WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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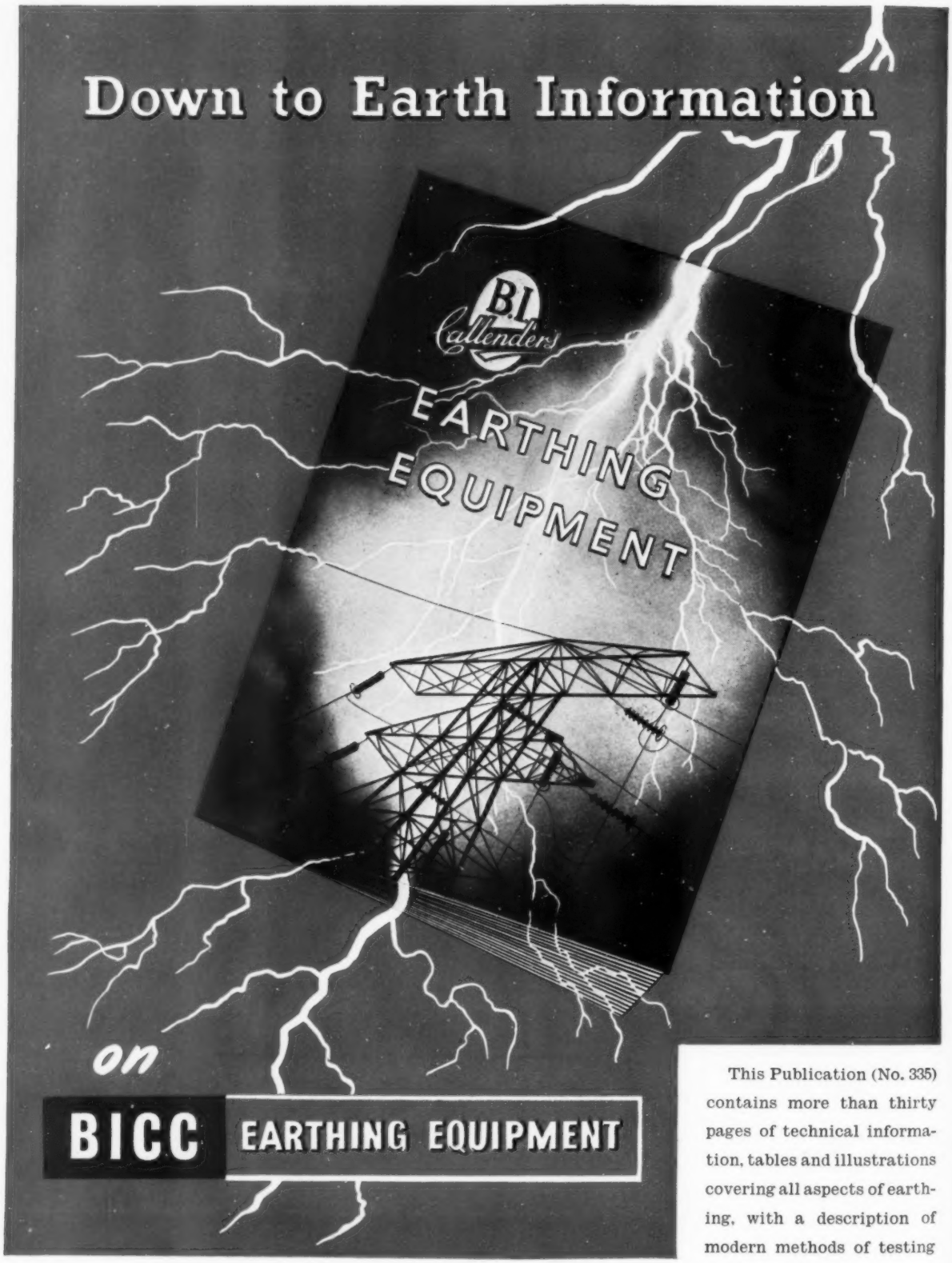

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

The Drop and the Atom

UNTIL recent times the smallest quantity with which a hydro-electric engineer had to concern himself was a single drop of water. Now it is becoming increasingly apparent that he must be concerned with an even smaller quantity—not only the atom but the electron. This little fellow is working his way into the hydro-electric power stations of the world with an ever-increasing rapidity. He quite naturally took his place in the communications that are essential in hydro-electric operation; this was his own proper field, and no heavy-current engineer felt any qualms about his activities in that direction. Next, perhaps, came the electronic specialist's claim to provide equipment of sufficient reliability to be used to control the governor gear of large water turbines. In this claim he has been largely justified; although we have heard water-power engineers say that they could not trust their plant exclusively to "those bits of wireless valves and gadgets." Nevertheless, the instantaneous nature of electronic control and the steady increase in valve reliability, coupled to the fact that the cost of such equipment is nowadays sufficiently reduced for it to be economical to duplicate every vulnerable part, is likely to lead to a steady increase in its adoption for frequency and voltage control.

Another form of electronic equipment which industry as a whole is using in a wide variety of ways, and which also finds application in hydro-electric power stations, is the use of artificial radio-active isotopes for detecting leaks in pipelines and glands. Here the Geiger counter is employed to detect minute traces of the radiations emanating from the isotope-laden water at the point where it escapes.

Electronic supervisory gear is likewise coming to the fore. Unattended hydro-electric stations, of course, are no novelty, but whereas the operation of such stations over the power lines by means of carrier equipment has always implied that everything "fails to safety" as the railwaymen say, newer methods of electronic remote control bring the valve equipment into more intimate association with the actual operation of the plant. Photo-electric cells are used to measure water levels; vibration detectors, with their outputs electronically amplified, are relied on to give the only indication of smooth running; thermo-couples embedded in windings are used as the sole means to supervise the healthy state of the electrical side, again making use of amplifiers.

The latest addition to the array of thermionic valves to be found in hydro-electric power stations is the computer. The complicated load flow and water-level calculations required to give optimum operating conditions in the light of all relevant factors are now, in some cases, carried out almost instantaneously by the aid of computing devices containing not scores, but hundreds of valves. The older school of engineers, in spite of the evidence provided by the length of life of valves employed for such duties as submerged telephone cable repeaters, still cast a wary eye at the apparently fragile "bottle," which they cannot dismantle and maintain. Their arguments include the suggestion that the displacement of the lower grades of skilled labour in hydro-electric power station operation by valve-controlled equipment may be offset by the need for highly skilled electronic technicians to maintain the multiplicity of gear now required. They suggest, moreover, that in the event of trouble occurring at a remote hydro-electric plant, in emergency almost any type of mechanic or even intelligent labourer can be pressed into service to help to restore the supply; whereas only an electronic expert can diagnose a failure in an amplifier. These arguments are discounted by the trend in industrial electronic control to provide equipment which is automatically duplicated and equipped with relays so that if any one of the rack-mounted sections into which it is divided should fail, the spare unit is automatically brought into service and a signal is sent to the nearest attended control point so that in due course the faulty unit can be withdrawn by an unskilled worker (in the electronic sense) and a spare always kept in readiness can be simply plugged in, ready for immediate operation.

The peculiar virtues of hydro-electric power generation, when compared with power generation by

means of diesel engines, steam turbines, or nuclear power plants, are all based on an inherent simplicity. This simplicity applies both to design and to operation. It is natural, therefore, that hydro-electric engineers should be chary of impairing this simplicity, even to the smallest degree. Their approach to electronic control gear is naturally cautious, but they cannot forever act the part of King Canute and pretend that simply by stating that hydro-electric power stations will run without electronic equipment, that the electronic tide will obediently recede. It seems prudent, therefore, for those concerned with the design and operation of hydro-electric power stations to take a greater interest in electronic developments than hitherto, and they may well find that the electron, the smallest particle of matter, is even more fascinating in its behaviour than the smallest drop of water.

Kitimat in Production

THE second and last series of electric furnaces at the new aluminium centre at Kitimat has commenced producing aluminium, according to a statement made by officials of Aluminum Company of Canada Ltd. With two potlines now producing aluminium the first phase of the Kitimat smelter is in full production. The first pour of metal from the first line of furnaces took place at Kitimat on August 3, 1954, in the presence of a distinguished group, including H.R.H. The Duke of Edinburgh. That climax to three and a half years of construction work is now followed by the starting of this second line and brings production to the rated capacity of 91,500 tons per annum.

Owen Falls to be Completed

THE Uganda Electricity Board has decided to complete the Owen Falls station by proceeding with the installation of the four remaining 15 MW generating sets, bringing the total to ten. Originally four machines were placed on order, but the number was soon increased to six, and by the time the Queen opened the station last April two machines were running. The news of the decision to complete the station is particularly reassuring as at one time doubts were expressed whether the necessary load to justify the outlay on the project would be forthcoming. It is evident from this decision that such fears have been proved groundless. The Owen Falls development was described in the June and July issues of this journal.

Table Rock Dam Tenders

THE lowest tender for two 68,000 h.p. turbines for the U.S. Army Corps of Engineers' Table Rock project at Little Rock, Arkansas, has been submitted by The English Electric Co. Ltd. At $1,066,662 (£380,950) it is $143,463 (£51,236) lower than that of Newport News Shipbuilding and Dry Dock Company at $1,238,540, and is the only tender submitted by a foreign manufacturer. Other tenders are Allis Chalmers, $1,238,540; S. Morgan Smith Company, $1,316,205; and Baldwin-Lima-Hamilton Corporation, $1,350,000.

According to Defence Department officials, acceptance of the English Electric tender will depend on a re-evaluation of the specifications involved, which might lead to a revision by the Army engineers in the amount offered by The English Electric Co. Ltd. as a basis for a contract, and whether or not the "Buy America" Act will be applied. Under the Act, high American bids may be accepted where the amount involved is more than $25,000 and where a foreign bid is less than 25 per cent. below the nearest American offer.

Ffestiniog and Rheidol Schemes

THE British Electricity Authority is seeking the consent of the Minister of Fuel and Power to promote a Bill in the forthcoming Parliamentary session for the development of the Ffestiniog pumped-storage scheme and the Rheidol hydro-electric scheme in North Wales. If these schemes are authorised by Parliament, they will together provide some 349 MW of additional generating capacity in the area. The proposals put forward in 1952 for hydro-electric development in these areas were deferred owing to restrictions in capital expenditure. Since that time further investigation has been made of the possibilities, and the Authority considers that the revised schemes now proposed are a marked improvement on the earlier ones in regard to their effects on local amenities.

In its original form, the Rheidol scheme provoked opposition in one or two quarters in respect of the proposals to divert the head waters of the rivers Severn and Wye into the Rheidol catchment, and those of the Rheidol into the Dovey estuary. None of these features, however, is included in the scheme now proposed.

The legislation proposed would be on similar lines to that embodied in the North Wales Hydro-Electric Power Act, 1952, in respect of the preservation of amenities, and due regard would be paid to riparian interests by providing for the discharge of reasonable quantities of water below each of the points of extraction in both schemes.

The works of the proposed Ffestiniog pumped-storage scheme would be located near the village of Blaenau Ffestiniog and close to the mining district, where there should be no interference with the natural beauty of the area. The scheme would utilise the existing Llyn Stwlan as an upper reservoir, the storage capacity of which would be increased by a concrete dam 90 feet high and about 900 ft. long. The lower reservoir would be formed by another concrete dam 35 ft. high and about 1,800 ft. long immediately upstream of the village of Tan-y-Grisiau in the valley of the Afon Ystradau. Between the two reservoirs, which will probably be connected partly by a tunnel and partly by surface pipelines, there would be a head of approximately 1,000 ft. The power station, which would be situated on the edge of the lower reservoir, would contain six combined generating and pumping sets, each consisting of a water turbine, centrifugal pump and generator/motor, together with ancillary plant. Water storage would be effected by off-peak thermal power. It is envisaged that the annual output from the Ffestiniog scheme, estimated at some 300 million kWh, would be fed into a section of the 275 kV Supergrid through a new switching station which would be constructed adjacent to the pump-storage station.

The Rheidol hydro-electric scheme is situated about ten miles due east of Aberystwyth in Cardiganshire. The available head of water would

be utilised in two main stages providing a total electricity generating capacity of 49 MW. In the upper stage, a reservoir would be formed by means of a dam about 150 ft. across the Afon Rheidol near Nant-y-Mach. The reservoir would be connected by means of a tunnel and a short surface pipeline to a power station situated some two miles downstream of the dam and discharging into the same river.

The lower stage would comprise a small balancing reservoir formed by a dam across the Rheidol just below the upper power station, and connected by a three-mile long tunnel to a second power station discharging into the Rheidol, a short distance below the Rheidol falls. The water would be returned into the lower reaches of the river, and, in order to avoid disturbance of the river regime by the intermittent discharges, a regulating pond would be formed by a small dam, creating a head of which advantage would be taken to generate an additional 1,000 kW. Part of the 83,000,000 kWh per annum output from the Rheidol scheme would be fed into the national grid by means of a new 132 kV transmission line between Rheidol and Oswestry. The remainder would be used to supply local loads.

Hydro-Electric Works in Lebanon

THE results of a three years' investigation into the proposed scheme to build a hydro-electric works on the River Litany, having a capacity of more than 600 MW, have been presented by the American Point Four consultants to the Lebanese Government. The project, to cost more than 100,000,000 dollars, and to be repaid over a period of 40 years, will irrigate more than 50,000 acres and establish a series of industries during the next two decades. The Lebanese Government Cabinet decided in 1953 to implement the Litany River irrigation and electric power project without foreign financial aid, after previously asking the U.S. Point Four funds for a 25 million dollar grant.

European Heavy Power Equipment

IN its annual review for the year ended April 1, 1954, of deliveries and orders for heavy equipment for thermal and hydro power stations, the Organisation for European Economic Co-operation states that there was a recess in deliveries except for waterwheel alternators and transformers. Deliveries of waterwheel alternators exceeded forecasts by 136 MW and totalled 4,302 MW, achieving an all-time record. Only three countries delivered less than in the past. Producers intend to increase considerably the capacity they will make available for this production. For 1954 it amounted to 4,720 MW per year, and in 1955 manufacturers expect to be able to ship about 6,150 MW. So far, however, they have received orders totalling 3,723 MW only.

Shipments of water turbines of 10 MW and over did not quite reach the peak of 1952, but deliveries, amounting to 4,757 MW, fell short of forecasts by 134 MW only. Delivery possibilities are expected to rise from 6,040 MW in 1954 to 6,385 MW in 1955, and even to 6,843 MW in 1956. Orders so far booked for delivery in 1955 amount to half of the availabilities only.

With regard to transformers, on April 1, 1953, the output in the following twelvemonth was estimated to amount to 32,421 MVA, i.e. the same volume as in 1952, but it reached 40,396 MVA and thus follows the rapid expansion noted since 1948.

Discussing the general situation, the report is satisfied that in Europe power-plant business as a whole will improve. Available figures show, however, that the importance of the output of hydraulic equipment is decreasing in comparison with the production of thermal machinery. The demand for equipment for new hydro plant in Europe is diminishing and no significant change in this trend may be expected in the near future, since most of the sites permitting economic power production under present conditions have already been exploited. Other promising hydro-generation potentials, particularly those in Norway, so the report says, cannot be exploited to a much greater extent until the technical problems of power transmission to the centres of electric consumption in Central and Western Europe have been solved.

Combined Project for British Columbia and Yukon

POWERS are being sought to establish a $270 million hydro-powered metallurgical industry on the borders of Yukon Territory and British Columbia. The plan envisages an initial development of 660 MW capacity, with the possiblity of an ultimate expansion to 3,750 MW. This scheme has been conceived by the Northwest Power Industries Limited and Frobisher Limited, both of which are interested in mining ventures. It is proposed to divert the upper reaches of the Yukon and Alsek rivers into the Taku watershed by a series of three dams, all of which will afford storage facilities, and two tunnels having a combined length of about eleven miles. By these means a gross fall of 1,092 ft. will be created, affording a power potential of 1,950 MW. A second stage of development will divert the headwaters of the Nakonake into the Taku river, and this will add a further 1,125 MW to the capacity. A further dam and power station on the Taku river itself will bring the total installed capacity to 3,750 MW. The metallurgical industries will be centred on a suitable site at the head of the Taku fjord, to which the initial power station will be connected by a transmission line 45 miles in length.

Shawinigan Results

GROSS revenue from sales of electricity for the first six months of this year was 4·7 per cent. higher than in the first half of last year; total gross revenue was 4·6 per cent. higher. General, operating and maintenance expenses were 4·4 per cent. higher, and the amount paid for purchases of power was 37·3 per cent. greater. The provision for depreciation increased from $3,298,000 to $3,395,000.

The amount of firm power sold to large industrial consumers and other utilities during the first six months of 1954 was 1·7 per cent. higher than in the corresponding period of 1953. Sales to residential, farm, commercial, municipal and smaller power customers were 13·1 per cent. higher. Sales of secondary power were 39·3 per cent. lower, due to less favourable run-off and storage conditions in the early part of the year as compared with the exceptionally favourable situation last year.

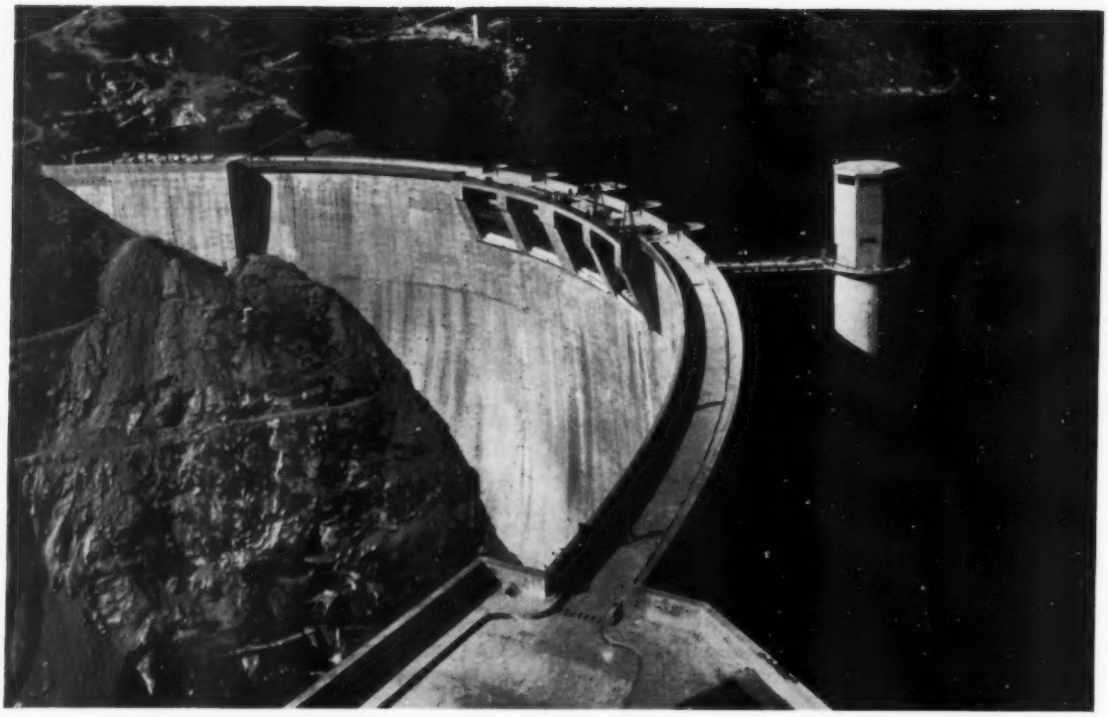


Fig. 1. Salamonde dam in May 1953

The Salamonde Hydro-Electric Scheme

Salamonde constitutes the second stage in the development of the Cavado and Rabagao rivers, one of the most important hydro-electric developments in Portugal. The ultimate scheme will have an installed capacity of 260 MVA and its output will equal the whole thermal and hydro output in Portugal for 1950. This article is particularly concerned with the civil-engineering design and construction at Salamonde.

By L. H. GOMES FERNANDES,
Chief Engineer of Hidro Electrica do Cavado

PART ONE

THE exploitation of the hydro-electric potential of the rivers Cavado and Rabagao in the northern part of Portugal was entrusted, by a Government Decree issued in January 1946, to the Hidro Electrica do Cavado, a limited company with its head office in Porto.

The main feature of this development is a catchment situated in a mountainous area with very high rainfall and presenting a comparatively high minimum rate of flow. All along the river, reaches of gradual slope alternate with steeper ones; the first are suitable for the construction of storage reservoirs whereas the latter provide large differences in head between the individual stages. The complete development at present under construction (Fig. 2) provides for the following schemes:—

(a) A dam at Venda Nova on the river Rabagao which creates a reservoir the water of which is diverted by a pressure tunnel to the Cavado valley and utilised by turbines in the Vila Nova power station;

(b) Three dams and a corresponding number of reservoirs in the Cavado valley. That farthest upstream, the Paradela dam, supplies other turbines in the Vila Nova station;* the other two, Salamonde and Canicada, have separate power stations.

Table I shows the main data of the various parts of the development and indicates its outstanding importance. With a total installed capacity of 260 MVA, the continuous annual production for the whole development consisting of four dams amounts to 920 million kWh for a year of average rainfall, equal to the total thermal and hydro-electric production of Portugal in 1950. A further small scheme on the upper Cavado, above the Paradela dam, together with a better utilisation of some small tributaries, will increase the continuous annual production in an average year for the entire development to more than 1,000 million kWh.

TABLE I

	Stage 1 Venda Nova	Stage 2 Salamonde	Stage 3 Canicada	Stage 4 Paradela
Height of dam ..... m.	97	75	76	112
Length of gallery ..... km.	3·5	2·1	7·8	10·5
Useful capacity of reservoirs ..... mill. cu. m.	92	55	138	156
Maximum total head ... m.	414	127	121	460
Capacity installed . MVA	90	50	60	60
Continuous annual production for an average year with all four dams in operation . mill. kWh	200	200	260	260

The Venda Nova stage was constructed first and began to operate in June 1951. Two years later the second stage, Salamonde, was opened in June 1953 in the presence of the President of Portugal and members of the Government. Work on the third stage, Canicada, is well advanced, the generators being in course of erection at the time of writing; it is hoped that production will start at the end of 1954. As far as the fourth stage, Paradela, is concerned, access roads totalling 25 km. have been completed, housing for workers and operatives is at present being constructed, and work has started on the diversion tunnel and the temporary river diversion.

The Salamonde Scheme

The outstanding feature of the Salamonde scheme (Figs. 5 and 6) is its underground power station situated immediately downstream of the dam, approximately 50 m. below the bed of the river and to the left of it. The main parts of the scheme are as follows:—

A dam 75 m. high creates a reservoir of 55 million cu. m. useful capacity the maximum water level of which has been fixed at elevation 280 m.

The water stored in this reservoir passes through a steeply inclined tunnel and drives two turbine-alternator sets of 25,000 kVA capacity each, installed in the underground power station.

An intake tower at the entrance to the pressure tunnel, total height 62 m., contains the operating gear for the tunnel gate.

Electrical energy is generated in the power station at 10,000 V and transmitted to the surface through busbars mounted on the walls of the access shaft. The voltage is raised to 150 kV by transformers installed in the outdoor substation which are connected to the transmission lines.

After passing through the turbines the water is returned to the river at elevation 153 m. by a tailrace tunnel 29·9 sq. m. in area and 1,860 m. long.

A surge chamber arranged immediately downstream of the station ensures the correct hydraulic operation of the system.

Location and Geology of Dam Site

The dam lies close to the town of Salamonde from which it takes its name, slightly upstream from the point where the Cavado is joined by a small left-hand tributary, the river Mau, and 5 km. downstream from its junction with the Rabagao.

The catchment drained by the Cavado and the Rabagao up to this point has an area of 623 sq. km.; it has an average altitude of 910 m. and a precipitation of 2,500 mm. in an average year. 240 sq. km. of this total area corresponds to the Venda Nova scheme on the Rabagao where another 228 sq. km. belongs to the future reservoir at Paradela, thus reducing the area draining directly into the Salamonde reservoir to 155 sq. km.

At the point chosen for the dam the bed of the Cavado lies at elevation 208 m., and the section across the valley is roughly symmetrical with respect to the centre of the river, some 30 m. wide at elevation 210 m. and about 230 m. wide at the highest normal water level in the reservoir, 280 m.

An exhaustive geological survey of the site was carried out by means of boreholes, trenches and tunnels driven into both banks of the river. It was found that the entire foundation of the dam would lie in coarse granite, slightly roseated in places. Although the rock is perfectly sound at the bottom of the river and in the lower parts of the banks, it changes gradually on both sides with increasing elevation. On the right-hand bank the granite outcrops in large blocks which have been eroded into more or less rounded shapes. The rocky banks are sound but contain numerous open fissures which close rapidly with increasing depth. On the left-hand bank the granite is almost completely covered with soil and dense vegetation and outcrops only in a few places; it is homogeneous with few fissures but considerably modified, the felspar being strongly decomposed. At a depth of ten or fifteen metres, however, the granite again becomes sound enough to form the foundation for the dam.

The elastic properties of the foundation rock were investigated not only on specimens collected from tunnels and borehole cores but also directly by tests carried out in the tunnels cut for the geological investigations. The purpose of these last tests was to determine the elastic modulus of the foundation material. Measurements were made in the vertical and horizontal directions both before and after cement had been injected into the rocks, the loads being applied by means of jacks. Table II shows the modulus of elasticity measured in each of the tunnels investigated, and as a comparison the values determined in the

*See WATER POWER August 1951, page 307, and December 1953, page 465.

laboratory on rock specimens collected in the same tunnels.

Design of Dam

Detailed investigations were carried out to determine the shape of the dam, both theoretically by the "trial load" method and experimentally on a small-scale model. They took into account the particular elastic properties of the subsoil, the assumed elastic modulus of the rock varying from 200,000 kg. per sq. cm. at the lowest point to 20,000 kg. per sq. cm. at the abutments.

TABLE II

	Tunnel tests with jacks				Tests on rock samples, 10 × 10 × 20 cm. prisms
	Before grouting		After grouting		
	Vertical	Horizontal	Vertical	Horizontal	
Tunnels, elevation 265 m., left bank	5 800	16 300	11 000	22 000	17 000 54 000
As above, elevation 250 m. - -	12 800	29 000	14 600	37 000	32 000
As above, elevation 225 m. - -	146 000	230 000	148 000	300 000	246 000 245 000
Tunnel, elevation 245 m., right bank	28 000	34 000	37 000	44 500	61 000 108 000
As above, elevation 260 m. - -	20 000	20 000	—	—	80 000 127 000

All values in kg. per sq. cm.

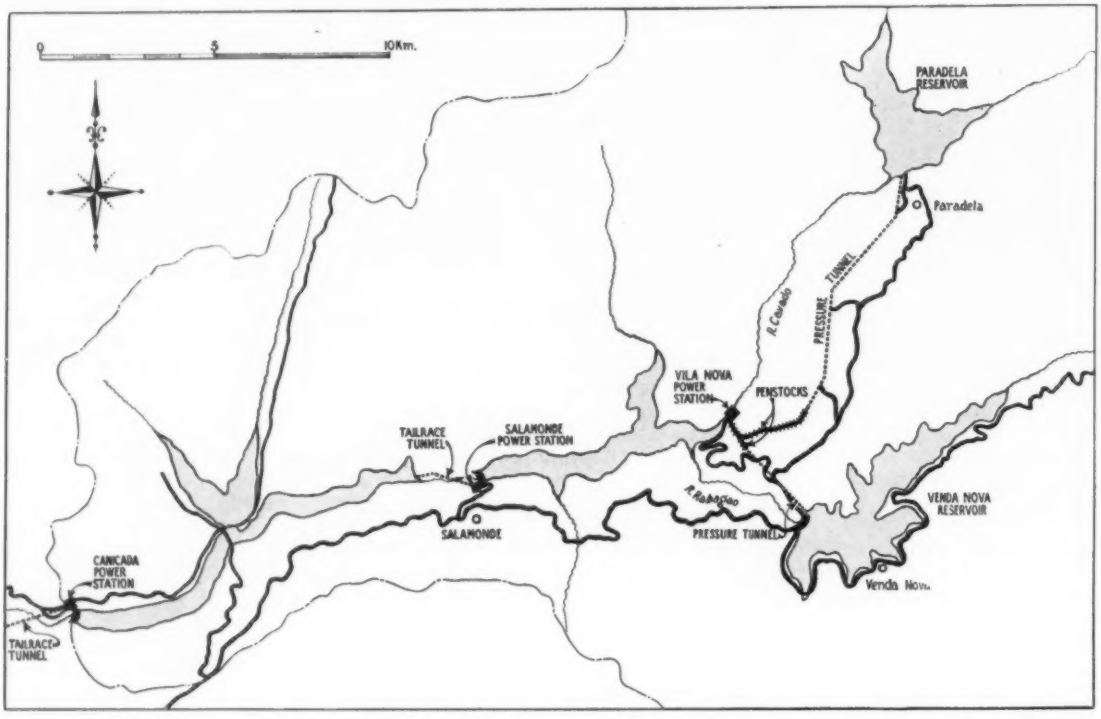


Fig. 2. Map of the developments on the Cavado and Rabagao rivers

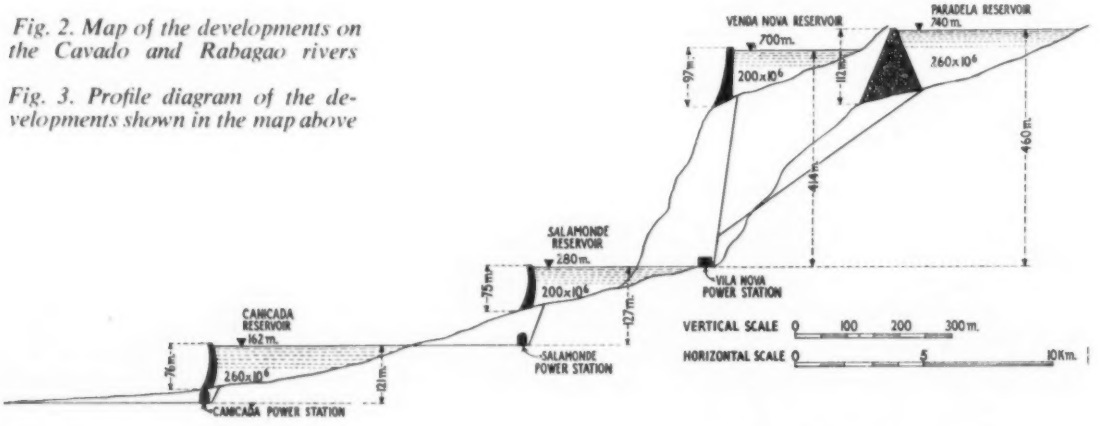


Fig. 3. Profile diagram of the developments shown in the map above

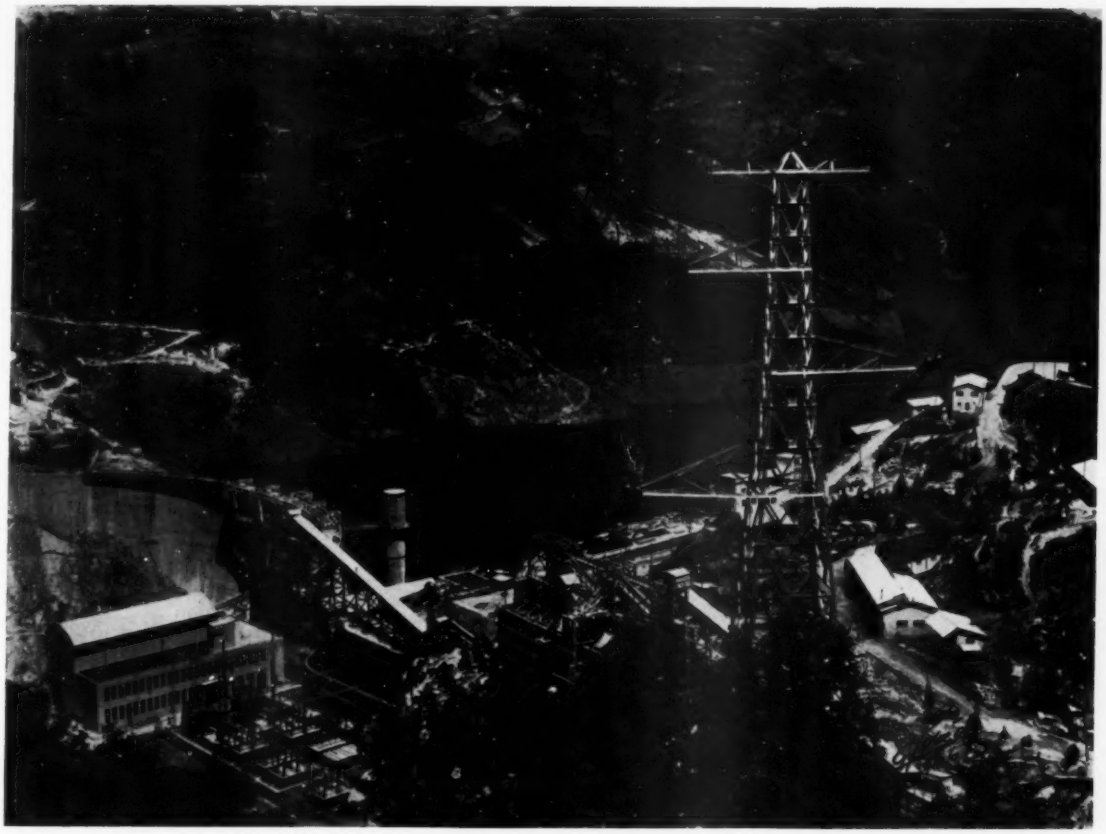


Fig. 4. General view of Salamonde development and construction plant. The surface buildings and switchyard can be seen at bottom left, and the crushing, grading and concrete-mixing plant upstream

The dam (Figs. 6 and 7) is a thin concrete arch, the radii of both upstream and downstream faces varying along their length. The lower part of the upstream face is strongly inclined towards the reservoir, making it necessary to construct several small buttresses on the upstream side to ensure the stability of the dam when the reservoir is empty. This inclination provides a means of reducing the vertical stresses due to bending at the base by utilising the compression in the opposite direction due to the deadweight.

In order to improve the load transfer to the foundations, a concrete base was constructed at the bottom of the centre dam section; it is so rigid that it can be considered not as part of the arch but rather as an extension of the subsoil. The main dimensions of the dam are:—

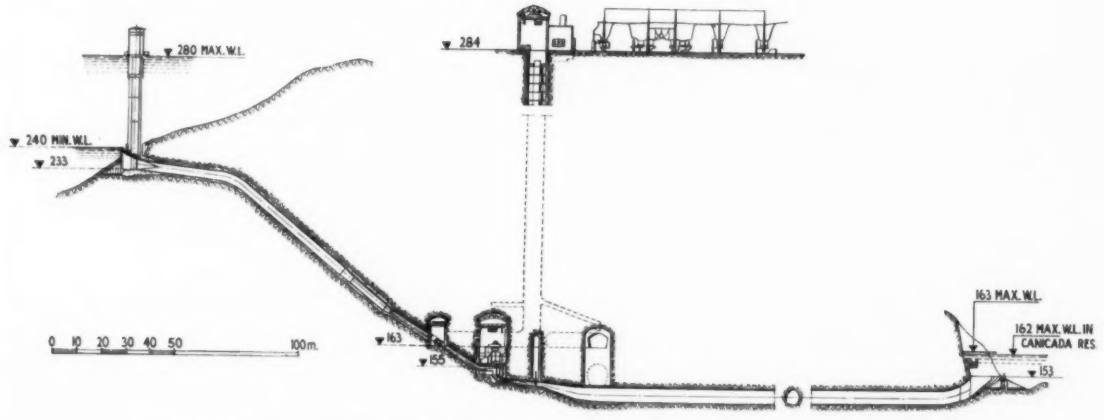


Fig. 5. Longitudinal section through Salamonde station and associated works

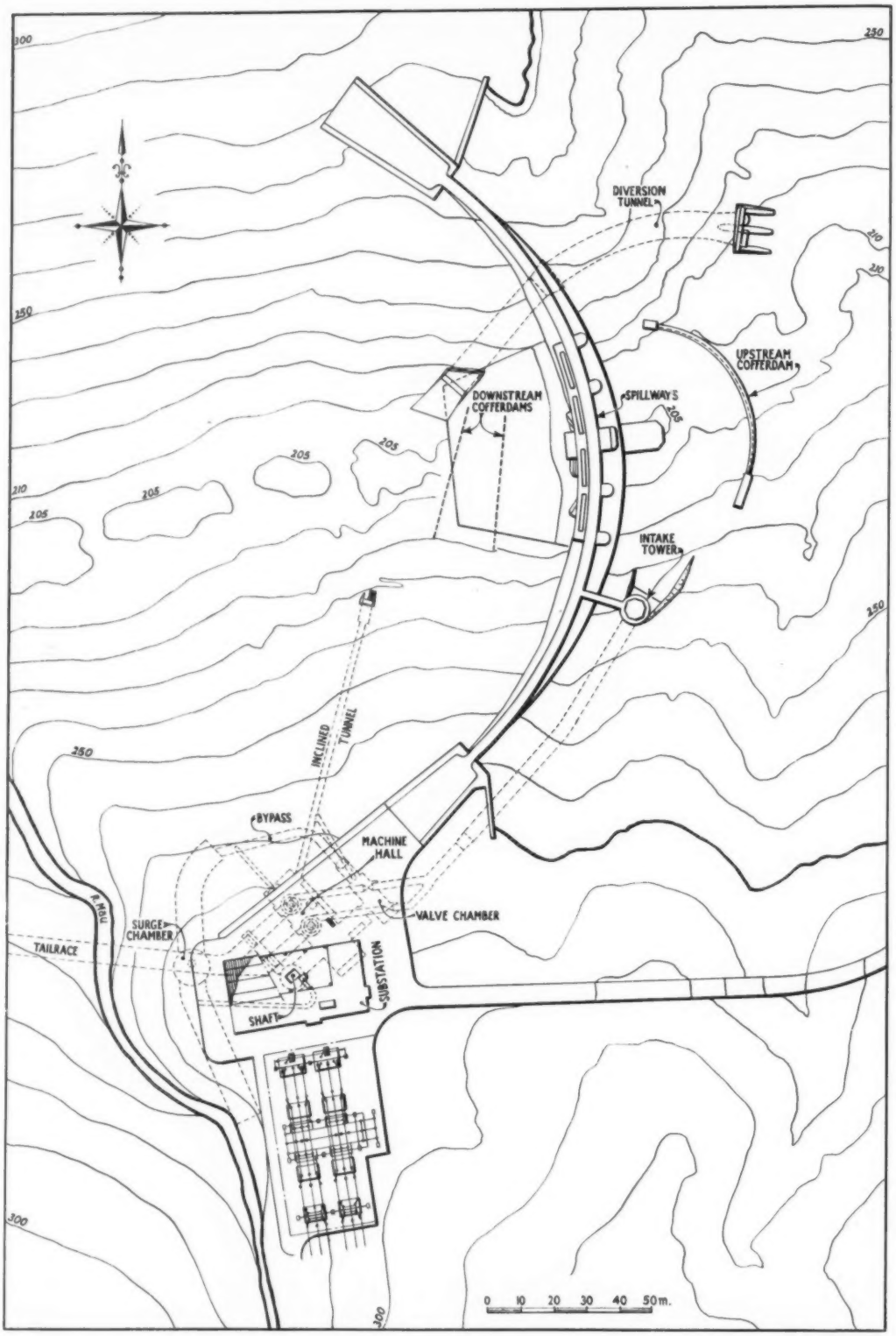


Fig. 6. Plan of Salamonde development showing the dam, cofferdams, surface buildings, switchyard, and underground power station

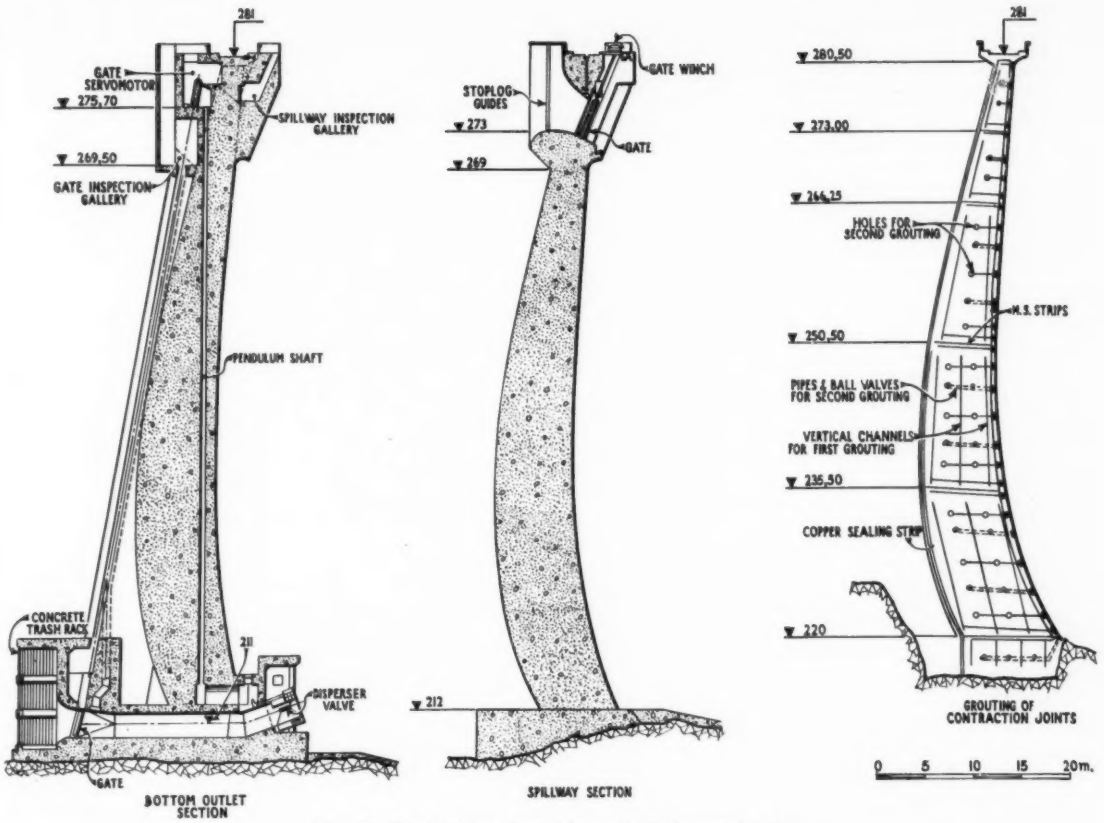


Fig. 7. Typical sections through Salamonde dam

Radius of upstream face at crest (el. 280 m.) ... 115 m.
Thickness at crest ..... 2 m.
Angle subtended by crest ..... 112°
Developed length between abutments .....203·3 m.
Thickness at base, at centre of arch .....6·35 m.
Maximum thickness ..... 10·6 m.
Maximum height above base ..... 70 m.

The shape of the arch surfaces is defined by the following equations in polar co-ordinates:—

(1) UPSTREAM FACE :

$$R_u = A + B \cos \theta$$

where $A = 115{\cdot}00 - 0{\cdot}25\ Z - 0{\cdot}1055\ (Z/10)^4$

$B = 0{\cdot}50\ Z + 0{\cdot}100\ (Z/10)^4$

(2) DOWNSTREAM FACE :

$$R_d = A' + B' \cos \theta$$

where $A' = 113{\cdot}00 - 0{\cdot}435\ Z - 0{\cdot}1325\ (Z/10)^4 - 0{\cdot}0007\ (Z/10)^6$

$B' = 0{\cdot}520\ Z + 0{\cdot}1300\ (Z/10)^4 + 0{\cdot}0007\ (Z/10)^6$

In these equations R_u and R_d are the radius vectors of the upstream and downstream faces respectively, Z is the depth below elevation 280 m., and θ is the angle at the centre, measured to either side of the centre-line of the dam, separately for each horizontal plane.

The calculations showed that the maximum stresses in the arch dam should be:—

	Compression	Tension
Maximum stress in arch ...	50·5 kg. per sq. cm.	2·0 kg. per sq. cm.
Maximum stress in buttress	36·0 kg. per sq. cm.	18·7 kg. per sq. cm.

Tests were carried out on a model of the dam which obviously did not take into account all the forces on the structure, such as changes due to temperature variations, swelling of the concrete and negative pressures; these forces had been included in the calculations, however. The results agreed on the whole sufficiently closely with the calculated stresses. Serious discrepancies were noted only at the base of the central buttress where the calculations gave a tensile stress of 18·7 kg. per sq. cm., as mentioned above, while the model tests indicated the presence of compressive stresses of the order of 47 kg. per sq. cm.

The load on the upper arcs of the dam is taken up by concrete abutments the stability of which is ensured by their deadweight. In order to relieve them of the hydrostatic pressure of the water in the reservoir, concrete cut-off walls were constructed which had been designed as small gravity dams. The soil between the abutments and the cut-off walls drains to the downstream side through holes in the abutments.

The crest of the dam is crowned by a roadway 3 m. wide at elevation 281 m., together with 0·75 m. wide footpaths and the appropriate breastwork. This latter is completely closed on the upstream side in order to prevent flooding of the crown if the reservoir level should accidentally rise above 281 m. On the downstream side the breastwork is also completely closed except at the spillway where it consists of a metal grid.

Construction Joints

The arch was constructed as 14 independent piers, the joints being approximately 14·5 m. apart. They are closed on the upstream side by copper strips embedded in the concrete of the adjoining piers. When the dam had been completed and most of the heat of hydration of the cement had been dissipated so that thermal equilibrium with the surroundings had been partly achieved, these construction joints were filled with grout in order to ensure that the dam formed a monolithic structure.

Provision was made for this injection of cement by closing the construction joints at their downstream side and dividing them into compartments by means of mild-steel strips. Grouting was carried out through small vertical channels arranged in the concrete faces along the joints; access to these channels could be obtained in each compartment from the downstream face.

In cases where further grouting proved necessary, these vertical channels could not be used after the first injections and one of the following methods was employed:

(a) Small ball valves were fitted on the surface of the joint, one for every 30 to 40 sq. m., and connected to a system of mild-steel pipes which in each compartment communicated with the downstream face of the dam;

(b) Asbestos-cement pipes were arranged to start from the downstream face and reach almost to the joint surface at certain selected points. For the second injection of cement into the joint it was only necessary to pierce the concrete with a pneumatic drill opposite the tube, followed by grouting with an ordinary valve.

The reservoir was first filled in May 1953 but only up to elevation 270 m. because the concrete in the upper parts of the dam had only recently been placed. Some time before this date the contraction joints were grouted up to elevation 265·5 m. (fourth compartment) at a pressure of 4 to 6 kg. per sq. cm. through the vertical channels mentioned above. A total of 22,900 kg. of cement was injected during this first phase of the work, corresponding to an average absorption of 6·6 kg. per sq. m. of joint surface.

It had been planned to fill the reservoir completely during the current hydrological year. The upper compartments of the construction joints which had remained empty were therefore grouted at the end of 1953, the reservoir level being lowered to 240 m. in order to reduce the load on the dam. The amount of cement absorbed by the two upper compartments above elevation 265·5 m. was 5,780 kg, corresponding to 4·8 kg. per sq. m. of joint area.

Concrete

The concrete from which the dam was constructed contained large-size aggregate up to 200 mm. diameter, produced by crushing and grinding granite obtained from a quarry close to the site the working level of which lies above the crown of the dam.

Fig. 8. The dam in October 1952 showing the river diversion works, upstream cofferdam, and the intake and outlet of the diversion tunnel

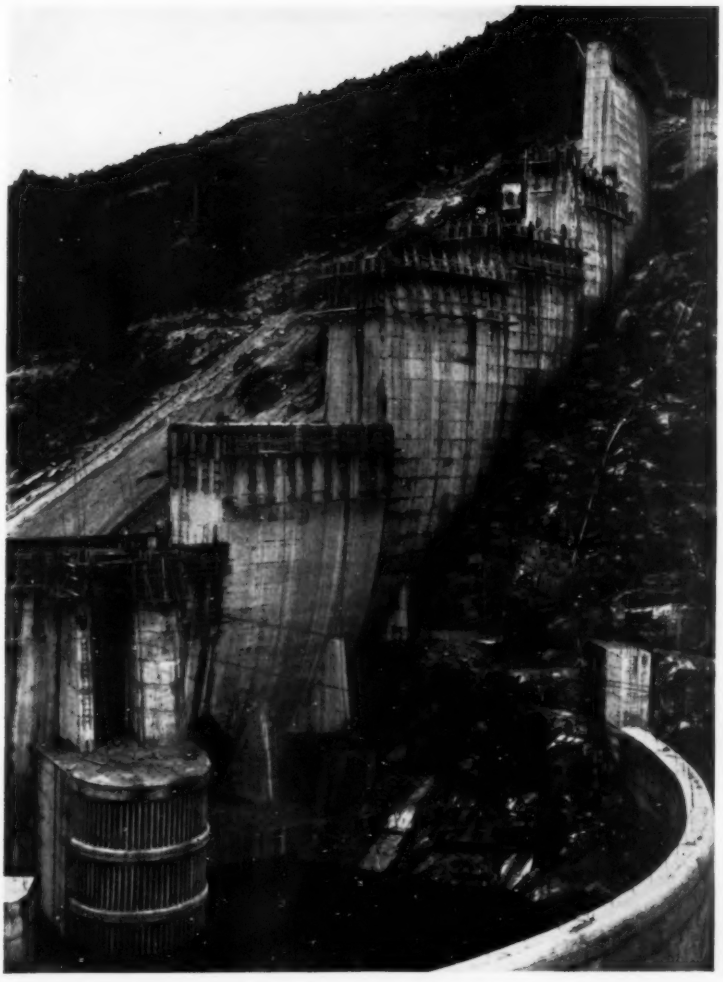


Fig. 9. Dam in October 1952, showing the bottom-outlet screen and the buttresses to support the dam when the reservoir is empty

The aggregate was graded into six sizes, namely

Medium sand	0 to 2·4 mm.
Coarse sand	0 to 4·8 mm.
Size No. 1	4·8 to 15 mm.
Size No. 2	15 to 30 mm.
Size No. 3	30 to 75 mm.
Size No. 4	75 to 200 mm.

The mix used had a discontinuous size distribution, sizes 1 to 3 being extracted and returned to the crusher for further reduction.

The concrete in the dam arch has a cement content when poured of 275 kg. per cu. m. where it is in contact with the foundation rock, 300 kg. at the lower parts of the upstream face, and 250 kg. in the remainder. At the abutments the initial cement content of 250 kg. per cu. m. was reduced gradually to 150 kg., an air-entraining agent, Darex Aea, being added in the proportion of 0·6 to 1·0 cu. cm. per kg. of cement in order to render the concrete workable.

The ratio of coarse aggregate to sand was always 3·3 irrespective of the quantity of cement, and the total amount of aggregate per cu. m. of concrete varied between 2,050 and 2,150 kg. The water-cement ratio corrected for the moisture content of the sand varied between 0·35 and 0·45; this gave a concrete with a 4 cm. slump when size 4 was omitted.

The sand was graded by the wet method in overflow cyclones, the following values of the fineness modulus being measured during construction:—

Medium sand	2·3 to 3·1
Coarse sand	3·0 to 4·6

The compressive strength of concrete containing 250 kg. of cement per cu. m., measured on 20 cm. cubes, was 140 to 150 kg. per sq. cm. after seven days, and 220 to 270 kg. per sq. cm. after 28 days; no concrete was used which, whatever its cement content, showed a strength of less than 120 kg. per sq. cm. after seven days.

The total volume of concrete in the dam, 96,000 cu. m., was placed between May 1952 and June 1953.

Ordinary Portland cement was used throughout and the characteristics obtained on site tests are given in Table III.

Grouting Curtain

The ground below the dam foundations was rendered watertight by a deep grouting curtain, the holes being sunk with a rotary drill from the bottom of the excavation through the foundation. The vertical dip of the main strata at the site suggested that the holes should be drilled into both banks at an angle of 45° and in a direction perpendicular to the river.

The number of grouting holes to be drilled was not decided on before work started. The holes were spaced initially 20 m. apart. If any 5 m. length of these holes under 10 kg. per sq. cm. pressure

TABLE III

	Values		
	mean	maximum	minimum
Compressive strength, measured on mortar cubes with 1 : 3 cement-sand ratio			
After 3 days...kg./sq. cm.	135	170	115
After 7 days...kg./sq. cm.	225	265	190
Heat of hydration. Maximum permissible after 7 days: 75 cal/g.			
After 3 days ..... cal/g.	53	62	45
After 7 days ..... cal/g.	62	70	52
Specific surface area, measured with Wagner's turbidimeter ..... sq. cm./g.	1680	1900	1460
Insoluble component ... %	0·90	1·21	0·79
Tricalcium aluminate ... %	12·4	14·0	11·0
Bicalcium silicate ..... %	34·2	41·0	23·0
Tricalcium silicate ..... %	37·8	46·0	32·0

and during 10 minutes absorbed more than 2 litres of water per minute per metre length of bore, adjacent holes were drilled at a distance of 5 m. The sinking of further holes depended in turn on these last holes, using the same criterion of permeability.

The minimum depth of the grouting holes was decided to be 20 m.; this, however, was increased until finally the water absorption of a 5 m. long section under 10 kg. per sq. cm. pressure during 10 minutes did not exceed 2 litres per minute per metre length of bore.

Cement grout prepared in Swibo mixers was injected into the holes through rising pipes at a maximum pressure of 25 kg. per sq. cm.

The complete grouting curtain determined on the basis described above involved drilling 60 holes totalling 1,557·4 m. in length; 426·7 tons of cement was injected, giving an average cement absorption of 274 kg. per metre bore.

In order to promote the absorption of the cement in those parts of the curtain, mainly in the central and upper region of the left bank, where the granite was homogeneous and not fissured, so that the cement was not absorbed or only with great difficulty, the grouting operation was preceded by the injection of a slurry of activated clay, having the following composition:—

Water	200 litres
Clay	50 kg.
Cement	2 kg.
Silicate	0·5 litres

This treatment gave excellent results and facilitated the penetration of the cement; altogether 46·2 cu. m. of this clay gel was injected.

It was noted that there was no connection between the water absorption during permeability tests and the amount of cement absorbed, so in order to check the effectiveness of the curtain, six further bores totalling 192·1 m. were sunk and treated similarly with activated clay. They absorbed 102·5 tons of cement and 99·5 cu. m. of clay, a mean absorption of 533 kg. per metre bore.

Grouting was also carried out on the upper left-hand half of the dam before the reservoir was filled, in order to consolidate the ground. Starting from the downstream side of the dam, 12 holes were drilled totalling 177·4 m. in length of which the first 5 metres were grouted at 25 kg. per sq. cm. and the remainder at 40 kg. per sq. cm. Immediately before injection these holes were treated with a solution containing 45 litres of silicate to 100 litres of water; they absorbed 102·8 tons of cement, about 580 kg. per metre bore.

After the reservoir was filled in June 1953, a slight seepage of water was noted at several points downstream of the dam on both banks. Sixteen boreholes were sunk with rotary and percussion drills, totalling 434·3 m. in length, and injected with 24 tons of cement and 45·6 cu. m. of activated clay.

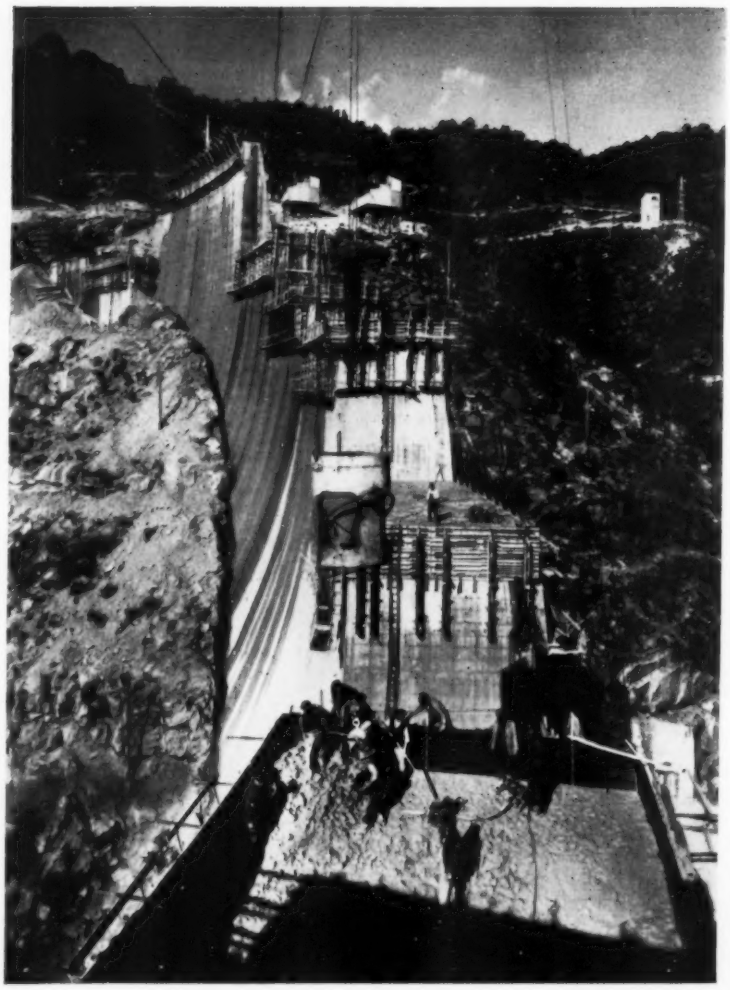


Fig. 10. Pouring concrete on the dam in October 1952

In order to ensure that the concrete adhered to the rock foundation, grouting holes were also drilled along the upstream side of the dam where it intersects the rock, spaced at 3 m. intervals and penetrating 3 m. into the ground. These 89 holes totalled 800·7 m. and required 19·4 tons of cement at a pressure of 5 kg. per sq. cm., giving an average absorption of 61 kg. per linear metre of contact between concrete and rock.

Deformation of Dam

The behaviour of the dam after construction and in service can be determined by measuring the internal stresses within the body of the concrete, as well as the deformation of the arch.

These stresses are measured by groups of Galileo vibrating-wire extensometers which were embedded in the concrete at suitably chosen points. A vertical shaft of elliptical cross-section rises from a chamber above the bottom outlet of the dam; a pendulum can

be suspended inside to measure the deformation of the arch. In addition, fixed points have been established along the dam to permit check surveys to be made.

These last measurements were carried out in July 1953 with the water in the reservoir at elevation 270 m.; they could be compared with similar measurements made two months earlier when the water level was at 245 m. It was found that the centre of the dam had moved 17·1 mm. downstream at elevation 260 m., while half-way between the centre and the abutments the deformations at this level were 6·5 and 7·2 mm. in the same direction.

A precision survey of the dam crest showed that the whole of it had lifted over the same period, the rise reaching a maximum of 3·8 mm. at the centre.

Reservoir Gates

The water in the reservoir can be discharged either through a bottom-outlet gate or by the overflow spillway. The bottom outlet consists of a metal conduit of 2·60 m. internal diameter located within the base of the dam at elevation 211 m. It is fitted on its upstream side with a $2{\cdot}0 \times 3{\cdot}0$ m. caterpillar gate and carries a disperser valve at the downstream end.

The machinery for operating this valve is installed in a watertight chamber near the outlet of the conduit and provides for both local and remote control, the latter from the control room of the scheme, located in the substation building.

The caterpillar gate is protected by a concrete trash-rack with a spacing of 20 cm. between its bars; it is operated by an oil servomotor housed in a watertight chamber in the centre pier of the spillway. For inspection, maintenance and repairs the gate can be raised along suitably arranged guides to a platform at elevation 269·5 m. The gate operating gear, like that of the disperser valve, can be controlled both locally and remotely; it also incorporates a device which automatically raises the gate to the open position as soon as any oil leakage from the piping allows the gate to fall a few centimetres.

The air inlet to the bottom-discharge conduit is connected to a 1 m. diameter pipe which rises to the crest inside one of the guides for the gate.

When the reservoir is at its highest normal level, approximately 130 cu. m. per sec. can be discharged through the bottom outlet.

The overflow spillway is arranged at the centre of the dam crest and consists of four rectangular openings $10 \times 4{\cdot}2$ m. the sill of which is at elevation 273 m.; they are fitted with Stoney gates operated by winches from the crest of the dam. The water discharges in the form of a free jet and falls on to a concrete carpet protecting the river bed and the lower parts of the banks.

The gates can be controlled either locally or from the station control room in the substation building; the position of the gates is reported at both points by special indicators.

The piers surrounding the spillway openings are

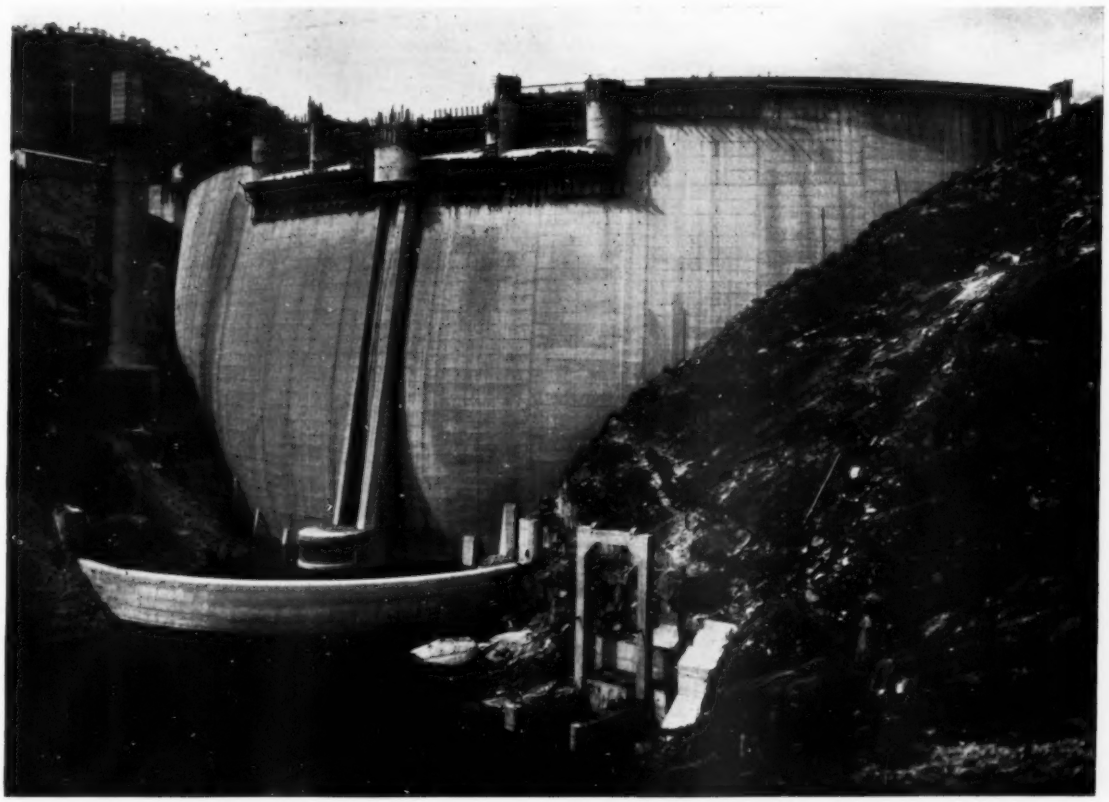


Fig. 11. The dam in April 1953. The spillway is under construction, the guides to lift the bottom outlet gate can be seen on the dam face, the diversion tunnel, already closed, is to the right, and the intake tower is on the left

designed to resist the load transmitted to them by the gates. Stoplogs may be fitted on the upstream side so that the gates can be unwatered when necessary for inspection, maintenance and repair.

The beams spanning the spillways and the sills were designed to take up without excessive loading the concentrated forces in the upper arches of the dam due to the spillways interrupting their continuity.

The maximum flood discharge predicted, 2,000 cu. m. per sec., is reduced to about 1,700 cu. m. per sec. by the reservoir rising to 0·52 m. above its highest normal level; the spillway has been designed for this reduced flow.

(To be continued)

Deflecting Young Salmon Downstream

A special correspondent describes Canadian experiments which indicate that a series of suspended chains may be the most effective means of deflecting sockeye smolts past hydro-electric works.

THE problem of getting young salmon—fingerlings or smolts—safely past power dams on their migration to the sea is a major concern on many waterways. Salmon are an economic as well as a sporting asset to many large rivers, and provision has to be made not only for their ascent, by salmon-ladder or Borland "lift," but also for steering them safely from the turbine intakes on their descent.

Three biologists of the Fisheries Research Board of Canada have recently studied the practical values of half-a-dozen methods of deflecting the descending salmon to the desired course. A large experimental trough was built at Lakelse Lake, British Columbia, in order to carry out adequate tests on the fish-guiding values of light, sounds, air bubbles, chain deflectors, odours and dyes in the water. Suspended chains gave the best results and further experiments are being made with this method. Salmon fingerlings were released, 800 to 1,000 at a time, from special release pens at the upper end of the experimental trough. This trough, a wooden construction, measured 58 ft. by 10 ft. and was 3 ft. deep, with metal screens at either end to permit the water to flow through.

The salmon used in the experiments were all young Pacific sockeye salmon, better known in Britain as tinned salmon, but responding much the same as Atlantic salmon. There are, however, some differences in salmon behaviour when in schools, e.g. between coho, spring and sockeye Pacific salmons. It appears to be a character of sockeye salmon schools to flatten in one plane only, and while the fingerlings would not converge to pass through vertical slit openings (5 ft. deep and 2 in. wide) they would readily enter these slits placed in the horizontal plane. Thus a series of vertical slits formed by chains or air bubbles might act as a deterrent to fingerlings.

The experimental use of light deflectors employed four 60 W white bulbs arranged in a linear series in a light box, placed either diagonally across the trough to give a vertical band of light, or in line with a glass divider to illuminate one half of it. Intensity was varied by a transformer, and colour by the use of amber and blue filters at 18·5 and 5·0 foot-candles respectively. Results varied. Little if any deflection was obtained with shining a narrow beam diagonally across half the trough; but the salmon noticeably avoided it when one half the area was totally illuminated. Intensity made no difference until it dropped below 10 foot-candles, when the effects deteriorated, and 0·2 foot-candle had no effect on the fish. Neither of the colour filters had any effect.

No effect was gained by trying to deter the fish with odours produced from human hands or the skins of certain fish which prey upon the young salmon. Bursts of deep methylene-blue dye ejected under pressure through perforated copper tubing every five seconds were good deflectors during the daytime, but not very good at night. No deflection could be produced by day or by night when sound was carefully employed at 1 to 10 kilocycles from a cylindrical transducer "beamed" at right angles to the long axis of the trough from a place inside and near the wall and remote controlled. During daytime, but much less at night, successful deflection was obtained by producing a rising screen or wall of air bubbles forced at pressure to rise through perforated copper tubing, having holes of $\frac{1}{32}$ in. at $\frac{1}{2}$ in. intervals and so placed on the trough floor as to screen one side of the dividing area. Even after 11 p.m. it deflected 58 per cent., but no such deflection was obtained with young "spring" salmon, a different species with different schooling behaviour.

The most successful result—between 94 and 98 per cent. deflection—was obtained by hanging small chains of $\frac{3}{16}$ in. by $\frac{1}{2}$ in. links, at intervals of 2 in., vertically in a wooden frame, to undulate gently in the flow of the trough. It worked on the principle that sockeye schools would not pass through vertical slit openings although they would pass through similar slits placed horizontally. Even at very low light these chains deflected the sockeye fingerlings, and at night low-frequency vibrations from the chains may have deflected the fish. It is suggested that migratory fish might be collected sufficiently far upstream above a high dam to be free from the accelerated flows of spillways and penstocks, and then guided into large side intakes in order to bypass the dam.

Even at night the effectiveness of a hanging chain wall fell only to an average of 71 per cent. When the interval between the chains was extended to 4 in. there was a slight lowering of its effectiveness, which became more marked when the spaces were increased to 6 in. On the other hand, coho salmon were inconsistent in their responses, sometimes showing no deflection at all. It seems that by mechanically vibrating the chain, increased deflection is obtained.

For the young sockeye salmon, which forms a very large proportion of the Pacific American salmon populations, the screen of agitated chains seems to be the most promising deflector to keep them away from high dams and intakes as they come down from their breeding haunts in the higher waters.

Surges and Waves in Open Channels

In this article a theoretical method of calculating the headrace surge wave which follows load rejection has been checked by full-scale experiments.

By E. T. HAWS, M.A., A.M.I.C.E.

ANY state of steady flow in an open channel, whether uniform or non-uniform, is affected by changing conditions at its ends. Whatever the nature of the change at the upstream or downstream end of the channel, its effect travels away from the source of the change as a wave. Travelling with velocity of propagation corresponding to the water depth, the wave effects a change in flow conditions at every point it reaches. This velocity of propagation is usually considerably in excess of the water velocity at steady flow. The new regime at any point is thus established before water from the source of the change reaches that point. Any gradual change in discharge alters the water profile slowly by a series of very small waves without conspicuous phenomena.

Such slow or small changes in flow in power installations occur at slow channel intake-gate movements and at small variations in machine demand.

In hydro-electric head or tailrace channels large changes in discharge can occur suddenly. These are accompanied by positive waves if the water depth increases, or negative waves if it decreases. These waves may be further subdivided according to whether the source of the change is upstream or downstream.

The positive waves are abrupt, or in the form of a train of undulations, depending on the depth of water on which they are propagated (Fig. 1). In the case of a long train of waves the second crest may slightly exceed the first.

The negative waves are sloping, and are often only marked by the lowering of the water surface.

When a positive surge wave passes any point the water level at that point does not remain stationary, but continues to rise. This rise is such that the longitudinal water profile at any time is that required to maintain the new flow conditions at any section, allowing for friction losses. The steady rise behind the surge wave continues until cut off by a negative wave.

The design of a hydro-electric headrace channel must allow for the surge following a maximum load rejection, considering both the initial surge crest and the steady rise at any point. The surge may be controlled by heightening the channel walls to contain the maximum wave and steady rise, by suitable spillway provision, and by automatic shutting of the channel intake gates. The surge wave (or waves) in a normal channel is of the "long" type, with wave length exceeding four times the water depth. For such waves the peaks travel slightly faster than the troughs, and so the wave cannot be propagated without change of shape. In a trapezoidal channel the wave amplitude increases in the shallow water at the edge. The change in shape there is exaggerated, and the wave may steepen to the point of breaking. The waves therefore appear hollow in the centre of the channel (Figs. 2 and 3).

The theory developed determines the mean surge height without allowing for peaks and troughs of individual waves. Shoklitsch[1] suggests an allowance of 50 per cent. above the theoretical surge height for the maximum crest, and this is probably sufficient for a rectangular channel or the centre of a trapezoidal channel. Any overtopping at the edges of a trapezoidal channel would be in the nature of a series of splashes.

At bends the waves pile up on the outside edge. The inside edge tends to run ahead and break (Figs. 2 and 3).

At solid surfaces, such as closed gates, a positive wave is reflected as a positive wave and a negative wave as a negative wave. The reflected waves are usually much smaller and pass back and forth along the channel until stable conditions are reached. A negative wave to cut off the steady rise behind a surge wave may be initiated by intake-gate closure, by reloading a machine in the case of load rejection, or by the collapse of the surge wave at the reservoir or spillway. Negative and positive waves travelling in opposite directions and meeting, cross and continue on respectively higher and lower water surfaces, irrespective of their relative size. Their size and velocity of propagation are altered only by the new depths and velocities of the water over which they are travelling. Thus a surge wave travelling on over water just lowered by a negative wave proceeds with less decay than previously, because of the reduced surface slope against it.

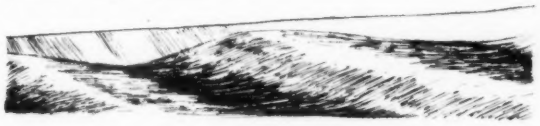


Fig. 1.

Fig. 2

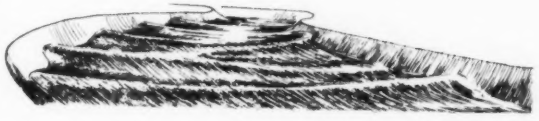


Fig. 3

Reloading a machine after a load rejection cannot stop the steady rise behind the surge along the whole length of the channel, unless the new flow demand equals the rejected flow. If the new flow demand is less than this amount, the steady rise will be stopped as far upstream as that point at which the flow behind the surge equals the new flow demand. The negative wave will continue upstream and check but not halt the rise at upstream chainages.

The case of the backwater surge due to stoppage at the downstream end of the channel will be considered in detail, as load rejection surge levels are usually the deciding factor in determining the wall height of hydro-electric headrace channels. The flood surge due to increased inflow at the upstream end yields to similar treatment.

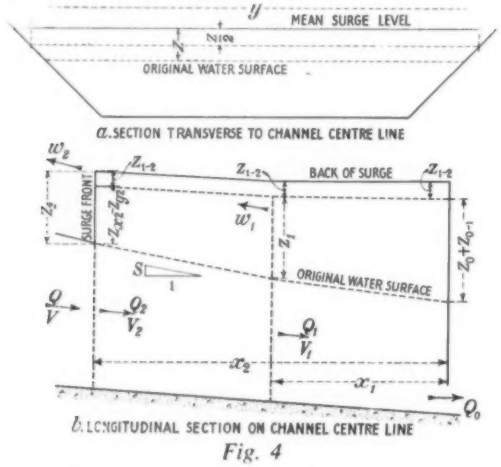


Fig. 4

The Backwater Surge* (Fig. 4)

NOTATION

Let A_f = the entire cross-section under the surge at its front.

V_f = the mean velocity in A_f.

Q_f = the discharge through A_f.

R_f = the hydraulic radius of A_f.

A = the original channel cross section.

V = the original mean channel velocity.

Q = the original mean channel discharge.

S = the original slope of the water surface.

D_m = the original hydraulic mean depth = $\frac{\text{area of flow}}{\text{top width of water surface}}$

z = the height of the front of the surge above the original water surface at any point.

z_0 = the height of the front of the surge when the front is at the beginning of the channel (i.e. $x=0$).

y = the width of the front of the surge at one half of its height, at any point, corresponding to z.

y_0 corresponds to z_0.

w = the speed of propagation of the surge at any point.

z_{1-2} = the amount by which the back of the surge rises throughout its whole length while the surge front travels from x_1 to x_2. It is a function of time only, and is independent of x.

w_0 = the speed of propagation of the surge at the beginning of the channel.

A_1 = the entire cross section under the surge at its front when the latter is at x_1.

Q_1 = the discharge through A_1.

Q_0 = the new outflow at the downstream end of the channel.

g = the acceleration of gravity.

ρ = the density of water.

Initially, $Q=AV$; when the wave reaches any point, one part, wyz, extends the step upstream with speed w, and the remainder, $V_f\ (A+yz)$, flows under the step to raise the back of the surge and provide the residual outflow Q_0.

$$AV = wyz + V_f(A+yz) \qquad (1)$$

The difference in static force between the two sides of the step is $\rho g\left(Az+\frac{yz^2}{2}\right)$ acting in an upstream direction, and retarding the inflowing water. Consider a transverse element of volume of water initially at a distance of $(V+w)$ upstream from the step; at the end of a unit time period, it will be under the step, and, therefore, the mass undergoing deceleration from V to V_f is $\rho A(V+w)$.

Then mass × deceleration = Force

$$\rho A(V+w)\ (V-V_f) = \rho g\left(Az+\frac{yz^2}{2}\right) \qquad (2)$$

But, from (1), $A(V-V_f) = (w+V_f)\ yz$

$$\therefore\ \frac{(V+w)}{g}(w+V_f)\,y = A+\frac{yz}{2}$$

and $$w = \frac{-(V+V_f)}{2}+\sqrt{g\left(\frac{A}{y}+\frac{z}{2}\right)+\left(\frac{V-V_f}{2}\right)^2} \qquad (3)$$

Equating flows at the downstream end of the channel,

$$Q = Q_0 + w_0\, y_0\, z_0 \quad \text{or} \quad z_0 = \frac{Q-Q_0}{w_0\, y_0} \qquad (4)$$

The height of the step decreases as it moves upstream, because a part of the inflow is used to raise the back of the surge. It is assumed that the slope of the water surface at any point after the surge has passed remains constant with time, i.e. the back of the surge rises parallel to itself. This is not quite true, as the surface slope will actually tend to flatten as the water depth increases. For long or very rough channels the assumption of constant slope with time gives a closer estimate of the surge height than the alternative assumption of a horizontal water surface behind the surge. The latter assumption leads to a great simplification of computation, and it may be used safely for short or very smooth channels.

To determine the surface slope at any point after the surge has passed, Manning's formula is applied, with a correction for change of velocity head. It is sometimes convenient in computing flow in a channel to include losses at bends, transitions, etc., in a fictitious value of the coefficient n higher than the true friction coefficient of the channel surface. After the surge has passed velocities and losses at bends, etc., are low, and the true value of the friction coefficient should be used in the following calculations.

The slope of the water surface immediately behind the wave crest is $S+\frac{\partial z}{\partial x}$

$$\text{Then } V_f = \frac{1{\cdot}486}{n} R_f^{\frac{2}{3}}\left(S+\frac{\partial z}{\partial z}\right)^{\frac{1}{2}} \qquad (5)$$

*Adapted from Shoklitsch.

where $\frac{\delta z}{\delta z}$ is negative.

At the step, the flow is $Q_f = A_f V_f = Q - wyz$ (6)

Equating (5) and (6) $\frac{Q - wyz}{A_f} = \frac{1{\cdot}486}{n} R_f^{\frac{2}{3}} \left(S + \frac{\delta z}{\delta x} \right)^{\frac{1}{2}}$

(7) or $dx = \frac{dz}{\left(\frac{wyn}{1{\cdot}486\, A_f R_f^{\frac{2}{3}}} \right)^2 \left(\frac{Q}{wy} - z \right)^2 - S}$

$= \frac{k^2 dz}{\left(\frac{Q}{wy} - z \right)^2 - k^2 S}$

where $k = \frac{1{\cdot}486\, A_f R_f^{\frac{2}{3}}}{nwy}$

k may often be regarded as constant for the entire channel, but in channels of variable section or roughness it can be evaluated for individual reaches.

The integration of (7) between the limits $x = x_1$, $x = x_2$, and $z = z_1$ and $z = z_{r2}$ results in

(8) $x_2 - x_1 = \frac{k}{2\sqrt{S}} \log_e \left(\frac{Q/wy - z_{r2} + k\sqrt{S}}{-Q/wy + z_{r2} + k\sqrt{S}} \times \frac{-Q/wy + z_1 + k\sqrt{S}}{Q/wy - z_1 + k\sqrt{S}} \right)$

z_{r2} is a first approximation for z_2.

This approximation has to be corrected by z_{a2}, the change in velocity head in the reach 1 – 2.

By continuity, $Q_2 = Q - w_2 y_2 z_2$ - - - - - - (9)

and then $z_{a2} = \frac{1}{2g} \left(\frac{Q_2^2}{A_2^2} - \frac{Q_1^2}{A_1^2} \right)$ - - - - - - (10)

z_{1-2}, the steady rise of the back while the surge travels from 1 to 2, must be added to $z_{r2} - z_{a2}$ to give the total surge at 2.

Thus $z_2 = z_{r2} - z_{a2} + z_{1-2}$ - - - - - - - (11)

z_{1-2} may be found by equating volumes. The time for the surge to travel from 1 to 2 is $\frac{x_2 - x_1}{(w_1 + w_2)/2}$

In this time the excess of water flowing into the section at 2 over that leaving the channel is

$(Q - Q_o) \frac{(x_2 - x_1) \times 2}{(w_1 + w_2)}$

This excess is used in extending the surge from 1 – 2 and in raising the back of the surge by z_{1-2}

Therefore

$\frac{2(Q - Q_o)(x_2 - x_1)}{w_1 + w_2} = (x_2 - x_1) \left(\frac{z_1 + z_{r2} - z_{a2}}{2} \right) y_{\left(\frac{x_1 + x_2}{2} \right)} + \int_0^{x_2} z_{1-2}\, y\, dx$ - - - - (12)

The latter integral can be taken in long reaches with the appropriate average y.

Trial and error between equations (3), (9), (10), (11) and (12) is necessary for a complete solution, but, after the first step, preliminary estimates of z_{a2} and w_2 should be close.

At the time the surge front arrives at 2, the height of the surge at $x = 0$ is $z_0 + z_{0-1} + z_{1-2}$; at $x = x_1$ it is $z_1 + z_{1-2}$.

z_2 is the mean surge height and is subject to an allowance for the initial crest of at least 50 per cent.

Variable Channel Section

Transitions are very difficult of solution by the above rigorous treatment. They may be broken down into a series of short reaches, the mean section being considered constant for any particular reach. In many cases it is accurate enough to assume the complete change of section takes place suddenly at the mid-point of the transition, and use that point as the end of a reach. Sometimes an inspection of the problem can lead to a simple solution, as for a forebay enlargement.

With the very low velocities after the surge has passed it is certain that the forebay will assume an almost horizontal surface. The velocity of propagation of the wave is therefore calculated as above for both ends of the forebay, and thus the time of passage of the surge through the forebay is known. Assuming a level surface behind the surge the height of surge at the downstream end of the forebay is then obtained simply by equating volumes (Fig. 5).

$\frac{(a+b)\,x}{2} \times \text{average } y = \frac{x}{(w_o + w_B)/2} (Q - Q_o)$

The difference in velocity head z_a between A and B can then be calculated and $z_a/2$ added and subtracted from a and b respectively. The method involves some trial and error in finding w_B.

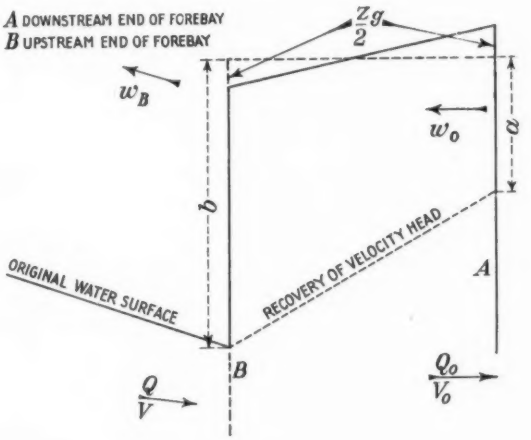


Fig. 5

Very Low Waves

For waves whose height is very small compared with the undisturbed depth of flow, equation (3) can be considerably simplified.

$w = \frac{-(V + V_f)}{2} + \sqrt{g\left(\frac{A}{y} + \frac{z}{2} \right) + \left(\frac{V - V_f}{2} \right)^2}$ (3)

If z is very small, V and V_f are sensibly equal.

$\frac{V + V_f}{2}$ becomes V; $V - V_f$ disappears

$\frac{z}{2}$ is negligible c.f. $\frac{A}{y}$

y becomes the top breadth at steady flow,

so $\frac{A}{y} = D_m$

$\therefore\ w = -V + \sqrt{g\, D_m}$

This applies to surge waves and negative waves. If

the wave is travelling downstream

$$w = +V + \sqrt{g\ D_m}$$

Generally, $w = \pm V + \sqrt{g\ D_m}$ - - - - - - (13)

This equation is used for determining the passage of the effect of small or slow gate movements, load changes, etc.

Slow Changes of Flow

The effects of these are propagated from their source as a series of very low waves. The water profile gradually approaches that required for the new flow. A method of obtaining the approximate profile at any time will be suggested, taking the case of a slow intake-gate closure.

Assuming a certain drop of water level immediately downstream of the gate as it closes, the quantity of water flowing through the gate at any particular opening can be calculated from simple orifice theory. At any particular time t after the gates start to close the quantity of water which has flowed into the channel is less than it would have been under the original steady condition by a certain amount R. (Fig. 6).

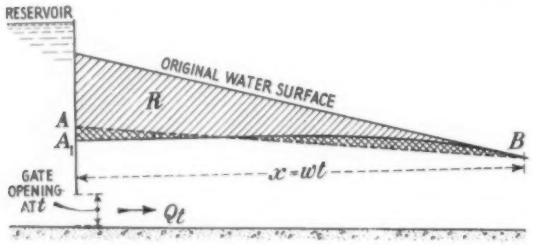


Fig. 6

At this time t the small wave initiated by the first gate movement will have travelled a known distance $x = wt$. Water in the whole of this reach will be affected by the closure. The plane surface AB can be computed to make the volume above it equal to R. The surface slope required to maintain the flow Q_t immediately downstream of the gates can be calculated, and the surface slope at B at time t must be that of the original water profile. A smooth curve can be sketched in between A_1 and B to make the double-hatched areas equal and the slopes at A_1 and B those required. If A_1 is the water level downstream of the gate assumed for time t, then the profile A_1B is approximately that required.

Superelevation of Water Surface at Bends

If the channel velocities at steady flow are considerable, it may be necessary to allow for superelevation of the water surface at bends.

A suitable formula is

$$h = \frac{2B}{r} \cdot \frac{V^2}{2g}$$

where h = superelevation on whole width,
B = top width,
r = radius to centreline of bend.

The whole of the foregoing theory has been fully confirmed by recent measurements made on a full-scale load rejection. The position and height of the surge wave throughout its travel in a long channel were predicted with remarkable accuracy, as was the effect of subsidiary negative and reflected waves.

[1] BIBLIOGRAPHY: Shoklitsch, "Hydraulic Structures."

New B.C. Construction Company

A new company has been organised by B.C. Electric to handle heavy engineering and hydro-electric projects all over British Columbia. The new firm, known as B.C. Engineering Co. Ltd., combines the present Engineering Division of B.C. Electric and the British Columbia International Engineering Co. Ltd., of 717 West Pender Street. The latter company, which comprises 40 engineers, designed and supervised the building of Cleveland dam and the gigantic Alcan development at Kitimat. For its part, B.C. Electric's Engineering Division has designed and supervised more than $200,000,000 worth of utility construction in eight years. Thomas Ingledow, vice-president and executive engineer of B.C. Electric, will assume additional responsibility as president of the new company. Jack H. Steede, executive assistant to Mr. Ingledow for nine years, will be chief engineer of B.C. Electric, and William C. Huber, general manager of the old B.C. International Engineering Company, will remain in that capacity in the new organisation.

Non-Ferrous Foundry Amalgamation. Two long-established Stoke-on-Trent non-ferrous foundries, T. M. Birkett & Sons Ltd., and Billington and Newton Ltd. have amalgamated. The company will be known as T. M. Birkett, Billington & Newton Ltd., and will be one of the largest non-ferrous founding firms in the country. The Board of Directors remains unchanged: A. P. Wright (Managing Director), J. C. M. Sheckell, A. Birkett, E. Y. Birkett and F. Bennet. Headquarters of the company will be a newly erected administrative block at Hanley which includes spacious office accommodation, drawing offices and a laboratory. The foundries and machine shops at both Hanley and Longport will, of course, remain in operation. Birkett, Billington & Newton make castings ranging in size from a few ounces to 10 tons in a variety of non-ferrous metals. One particularly interesting job completed recently was a turbine runner casting in aluminium bronze for a 3,600 h.p. Turgo impulse wheel. The specification called for a tensile strength of 45 tons per sq. in. and a high resistance to erosion, corrosion and abrasion.

Orders for Small Turbines. One of New Zealand's isolated tourist resorts, Milford Sound, is to have a hydro-electric generating unit. The turbine for this unit is being built by Gilbert Gilkes & Gordon Ltd., Kendal, and consists of a 635 b.h.p. Pelton wheel running at 750 r.p.m. on a net head of 725 ft. Other orders received recently by this firm are for a 750 b.h.p. Francis turbine working at 750 r.p.m. on a net head of 120 ft. and a 320 h.p. Francis turbine working at 1,000 r.p.m. on a net head of 117 ft. These turbines are for Nyeri, Kenya and Nelspruit, South Africa, respectively. A municipality in the Province of Quebec has also placed an order for a Francis turbine, to develop 600 b.h.p. at 900 r.p.m. on a net head of 165 ft. The demand for instructional turbines for college laboratories continues to increase, and a relatively large number of these units are in hand at the moment, a large proportion being for universities and technical colleges in India and Pakistan.

World Power Conference

Short abstracts are given of the papers relating to water power that were presented at the Brazil Sectional Meeting of the World Power Conference.

PART TWO

THE importance attached to the correlation between thermal and hydro-electric generation is evidenced by the receipt of the texts of six further papers on this subject which were read at the recent Brazil Sectional Meeting. The following summaries should therefore be regarded as an addendum to our review of papers under this heading last month.

The Thermal Power Plant as the Complement of Hydro-Electric Developments in Regions of Abundant Hydraulic Potential. By Walter Dreyer, United States National Committee.

The initial interconnection of thermal and hydro plants took place in California in 1899 and demonstrated for the first time the benefits inherent in an integrated operation of thermal and hydro-electric plants.

The functions of thermal plants operating in conjunction with a hydro-electric system are:—

1. To effect the best system economy.
2. For standby at important load centres.
3. For meeting the seasonal hydro deficiency during the short water period on non-regulated streams.
4. For meeting the dry-year hydro energy deficiency.

Where available hydro is not adequate fully to meet these increasing demands, power generated by thermal means must be provided for this purpose as well. As far as possible steam plants should be located at important load centres so they can serve all of the aforesaid purposes.

The economics of "mingled power" are discussed and described with examples which demonstrate why and how in general power can be provided for less cost from a combination of thermal and hydro plants than from either all-thermal or all-hydro systems. Maximum economy is achieved with "mingled power" when the system is designed so that at times of system peak demand there will be utilised:

(1) The full installed capacity of steam-electric plants and hydro-electric plants other than run-of-river plants and those with variable head located at storage reservoirs; (2) the full dependable capacity of such run-of-river and variable-head hydro-electric plants; and (3) the dependable capacity available from other sources. To plan the power system in any other way will result in duplication of some amount of installed capacity, with its attendant continuing burden of fixed charges.

The paper describes how the load may be assigned to different types of plants with normal and also with deficient water supply in order to obtain full use of the available plant capacities. It also describes the necessity for reserves in such a system to meet the following purposes:—

(1) Rolling reserve immediately available to meet changes in local or system interruptions.
(2) Reserve for overhaul of generating equipment.
(3) Reserve for outages not planned or scheduled.
(4) Reserve for load growth in excess of estimates.

The paper concludes with a short discussion of the possible place for nuclear power plants in such integrated power systems.

The Thermal Power Plant as the Complement of Hydro-Electric Developments in Regions of Abundant Hydraulic Potential. By Richard L. Hearn, Canadian National Committee.

The Ontario Hydro developed a power system to meet the electrical needs of highly industrialised centres and good farming areas in the southern part of the Province of Ontario, Canada. The power resources of the system were almost entirely hydro-electric until about 1950. Since that time two major thermal-power plants have been constructed and an interconnection with a neighbouring thermal-power system established.

Only one major hydraulic site remains undeveloped in southern Ontario. Potential resources comprise additional thermal-power plants, undeveloped hydraulic power sites in northern Ontario, additional purchases from other agencies, and the possible production from nuclear reaction.

The Ontario Hydro assesses the capability of its hydro-electric developments on the basis of monthly mean flows occurring 85 per cent. of the time. Although the ratio of the capacity of continuous sources of supply to the total sources declined, no serious difficulty was experienced from adverse water conditions until 1947-48. During this period marked deficiencies in precipitation reduced the total energy resources some 17 per cent. below the estimated dependable output. This, combined with high loads, necessitated restrictions in the use of electrical energy.

Initially, thermal power plants were constructed to meet fast-rising load demands since they could be built with least delay. Additional thermal units were added to provide peak capacity and energy reserve against the effects of low river flows.

In 1953 Ontario Hydro again experienced the effects of adverse water conditions brought about by sub-normal precipitation which fortunately developed when system loads were relatively light. The reduction in output of the hydro-electric plants affected was offset by the increased use of thermal-power plants and the purchase of power from a neighbouring thermal-power system. Therefore, no restrictions in the use of power were necessary.

The paper points out that if plant capability is based on monthly mean flows available 85 per cent. of the time and the ratio of the variable sources increases compared to the constant sources, then the chances of a system energy deficiency become greater.

Austria: Various Types of Thermal Power Plants as Complementary Establishments to Hydro-Electric Plants. By Franz Hintermayer and Herbert Melan, Austrian National Committee.

The paper points out that the production of electric energy must be adapted to demand. Meeting this requirement proves to be particularly difficult in countries with predominant hydro-power production since the output of the various types of hydro power plants is in part or wholly depending on the actual water supply. It appears, therefore, necessary to compare an analysis of demand with an analysis of production in order to ascertain what portion of the available waterpower energy can be utilised and how much energy for covering demand is lacking (i.e. to ascertain the so-called energy deficit). As a criterion for evaluating the energy deficit to be covered by means of "bedarfsabhängige" power plants, i.e. by plants whose production is determined by the actual demand, the number of hours is used during which these supplementary plants are on line; a method for establishing these basic data is described and illustrated by an example.

As far as thermal power plants are to be considered as supplements to hydro power plants, they should be developed as heat installations or as combined gas-steam plants in order to ensure a high degree of operational economy through waste-gas utilisation. They should be equipped with heat pumps whose heating stage is used in the heat cycle of the heat carriers while the cooling stage serves for providing cooling media in the hot season for conditioning residential space, etc. It is shown that of all supplementary installations a gas or hot-air turbine plant best meets these requirements and that special significance attaches to it as a waste-gas plant. The thermic effects of the arrangements proposed surpass those achieved by the hitherto known types operated exclusively on back pressure.

Thermo-Electric Plants as a Complement to Hydro-Electric Development in Bombay-Poona Region. By The Hydro-Electric Power Supply Co. Ltd., Bombay, Indian National Committee.

The Tata Hydro-electric system in Western India exemplifies the use of thermal power as a complement to sizeable regional hydro resources. Coal is available at high cost after a long haul from Central India, favouring exploitation of hydro-electric power sites which are characterised by abundant rainfall and high heads. In spite of this, thermal power plays a vital role in the development of the region.

Steam power previously to 1915 built up load taken over by the first hydro plant at Khopoli station. Operation of another plant at Bhivpuri, built in 1922, shows how thermal power increases its output by 21 per cent. in dry years, by allowing full utilisation of its reservoir. A third station followed at Bhira, and after interconnection with the Great Indian Peninsular Railways grid in 1939, Chola power station was also incorporated to the system. A modern 100,000 kW steam plant at Trombay will allow load demand to grow until future two-stage 480,000 kW Koyna valley hydro-electric project goes into economical operation, and thenceforth will act as it did for Bhivpuri station. Until this latter stage is reached, hydro will operate at base load, thus reducing operational cost of thermal stations by decrease of fuel consumption.

Planning of New Power Plants into a Co-operative Power-Plant System with Particular Regard to Run-of-River Plants. By Andrew Lévai and Emil Sóváry, Hungarian National Committee.

Methods of calculation are given from which the following conclusions may be drawn:—

a. A run-of-river plant without storage, for instance the economical hydro plant to be developed on a Hungarian river, has a firm capacity nearly equal to that of a thermal plant having the same installed capacity, despite the fact that its generating capacity is subject to substantial fluctuations owing to variation in the run-off. This result supports the well-known tendency according to which hydro plants are developed in increasing number and for increasing discharges.

b. District-heating power plants using pass-out condensing turbines or a combination of heating turbines and straight condensing turbines are worth hardly less than some straight condensing power plants of nominally the same size, and will therefore play an increasingly important role.

The Joint Utilisation of Hydro and Thermal Electric Power in Japan. By Toshio Yoshioka and Kyuichi Yamazaki, Japanese National Committee.

Since before the war, Japan has followed the policy of designing hydro-electric plants for a maximum discharge three to four times the minimum, and of using thermal power to supplement during water shortages. This use of hydro power as the primary and thermal power as the secondary source of supply has arisen because hydro resources are comparatively abundant and indigenous fuel resources meagre. It has also been justified from the standpoint of cost. Numerous studies have shown that the most economical procedure during periods of water shortage is to use hydro generation for peak loads, keeping thermal power on base load, thus keeping to a minimum the thermal capacity it is necessary to install. During periods of abundant water thermal power should share in the peaks to decrease spillage. It is considered that it is most economical to fix the maximum discharge of hydro plants to one-and-a-half to twice the minimum flow, although generation cost rises only slightly when this figure is increased to three to four times the minimum flow.

Miscellaneous Papers

Three further papers on various subjects have also now been received:—

Development of International Water Powers in Europe. By Leonhard Wolf, National Committee of the German Federal Republic.

The harnessing of international waterways is steadily gaining in importance, as rivers are frequently involved which promise a highly economic exploitation. The problems raised in this connection are primarily those of private law, public law, and international law.

Of special significance regarding the development of international waterways are the rules of international law, in this field termed international water law. Here we find two principles opposing each other; the principle of territoriality grants each state an unrestricted right to dispose of its water powers without consideration for neighbouring states, and the principle of integrity declares that no state may, without consent of the neighbouring state, take any steps, or

allow such steps to be taken, which might lead to an appreciable inconvenience or impairment of that state. Attempts hitherto at arriving at multilateral conventions in the field of international water law have not matured into practical significance, but actual practice has shown that bilateral agreements on certain projects were possible in all cases involving given parallel interests. This is the rule in regard to waterways forming the boundary between two states (contiguous waterways) as in opposition to waterways bisected by the frontier (successive waterways).

The report deals mainly with the construction of individual power stations on the Inn and on the Danube as contiguous waterways and shows the solutions arrived at in recent times. In addition, constructions on successive waters are outlined. Finally, mention is made of present studies concerning the international development of national waterways.

Considerations sur la Conception des Centrales Hydro-Electriques en Pays Tropicaux et Particuliere-ment au Congo Belge (Considerations on the Design of Hydro-Electric Stations in Tropical Countries, with special reference to the Belgian Congo). By J. P. Zehnle, National Belgian Committee.

This paper begins with a brief reference to the general characteristics of the geography of tropical countries, namely:—

(a) from the point of view of physical geography, morphology, fauna, climate and hydrology.
(b) from the human aspect; population, communications, technical development and the general state of knowledge of the countries considered.

The influence of the above factors on the design of hydro-electric power stations is then considered in the following stages:—

(a) determination of the general features of the scheme, installed capacity, location of the works, characteristic flows.
(b) studies of the civil-engineering works taking into account servicing conditions and conditions of construction.
(c) study of the hydro-mechanical plant and equipment.

L'Inventaire Total des Ressources Hydrauliques comme Base des Plans Generaux de Developpement (A Complete Inventory of Hydraulic Resources as a Basis for General Plans of Development). By M. Rousselier, French National Committee.

By the use of convenient methods of prospecting and estimating, it is a comparatively easy task to compile a complete inventory of water-power resources to a very approximate limit of relative value. Such an inventory possesses an interest beyond that of knowing the extent of the resources, a sufficiently accurate estimation of which can only be obtained from a summation of effective projects. The analysis of economic and statistical data provided by the inventory affords wide scope for preparing a development programme, for organising investigations, and for following technical trends. Although such an inventory is valuable only if it is constantly reviewed, it forms an indispensable basis for planning. The results given in the paper are only for French Metropolitan Territory.

International Congress on Large Dams

The following is the list of nine papers and two "communications" to be presented by the British National Committee at the Fifth Congress of the International Commission on Large Dams of the World Power Conference, which, as already announced in this journal, will open in Paris on May 31, 1955:

Question No. 16: Design and Construction of Dams on Permeable Soils and Methods of Foundation Treatment.

1. "The Gain in Stability due to Pore Pressure Dissipation in a Soft Clay Foundation," by A. W. Skempton and A. W. Bishop; Imperial College of Science and Technology.
2. "The Treatment by Grouting of Permeable Foundations of Dams," by C. F. Grundy; The Cementation Co. Ltd.
3. "Stabilising an Earth Dam Foundation by Sand Drains," by G. A. R. Sheppard and A. L. Little; Binnie, Deacon & Gourley.

Question No. 17: Economics and Safety of Different Types of Concrete Dams.

1. "The Employment of Prestressed Technique on Allt-na-Lairige Dam," by J. A. Banks; Babtie, Shaw & Morton.
2. "The Design and Economics of Massive Buttress Dams," by E. J. K. Chapman and D. Fraser Campbell; James Williamson and Partners.
3. "The Heightening of a Gravity Dam," by C. M. Roberts; Sir William Halcrow and Partners.

Question No. 18: Settlement of Dams due to Compressibility of the Dams Materials or of the Foundation Soil, including Earthquake Problems.

1. "Difficulty Experienced in the Construction of the Hollowell Dam, Northampton," by J. Kennard; Edward Sandeman, Kennard and Partners.
2. "Some Unusual Foundation Problems arising from Construction of Dams in Active Volcanic Country," by J. K. Hunter and H. G. Keefe; Sir Alexander Gibb and Partners.

Question No. 19: The Relation of the Cement Content of the Concrete to Performance in Practice of Gravity, Arch and Buttress Dams and its Influence on Permeability and Frost Resistance.

1. "Concrete Dams in Great Britain: Cement Content in Relation to Performance," by C. R. Lee; Building Research Station, D.S.I.R.

Communications

"Investigation of Stress Distribution in a Diamond Head Buttress Dam by Photo-elastic Model Tests," by H. G. Keefe; Sir Alexander Gibb and Partners.

"Investigation of Right Angled Bends at the Foot of Vertical Spillway Shafts," by Oscar Elsden; Sir Alexander Gibb and Partners.

Copies of Bulletin No. 1, issued by the French National Committee, are still available and may be obtained from: The Secretary, British National Committee on Large Dams, 201, Grand Buildings, Trafalgar Square, London, W.C.2. This Bulletin contains the programme and timetable of the Congress proper, information about the technical tours in France and North Africa organised in association with the Congress, and a provisional individual registration form.

Book Reviews

Hydro Power Engineering. By James J. Doland, D.Sc. Published by The Ronald Press Company, 15 East 26th Street, New York 10, N.Y., U.S.A. 9 in. × 6 in. × $\frac{5}{8}$ in. 209 pp., plus index. Illus. Price $7·50.

In this book Dr. Doland, who is Professor of Hydraulic Engineering, University of Illinois, and who has done conspicuous work for the National Resources Planning Board, the War Production Board and the U.S. Bureau of Reclamation, offers a concise and severely practical approach to water-power development. The scope of the book is restricted to practice in the U.S.A., and the first chapter outlines the general circumstances there in relation to the power market, the trend towards the total development of a river basin, and the respective parts played by private companies and the Federal Power Commission. Chapter 2 deals with the fundamentals of water-power development, covering such items as stream discharges, head, types and designation of turbines, turbine constants, specific speeds, and typical development schemes. The next chapter considers the analyses of data—stream flow, flow-duration curve, mass curve, firm power, load factor, pondage, storage, etc. Further chapters deal with hydraulic turbines and their preliminary selection; dimensions of water passages and power houses; gates, valves and penstock design; water hammer, surge tanks, trashracks, governing gear; the economic aspects of hydro-electric development; and hydro-steam association. The final section of the book is devoted to problems.

This book is a masterpiece of comprehensiveness and conciseness, but it necessarily has the defects of these virtues, and its empiricism may not suit some readers, though all will find it valuable.

What Every Engineer Should Know About Rubber. By W. J. S. Naunton. Published by The British Rubber Development Board, Market Buildings, Mark Lane, London, E.C.3. 1954. 126 pp. Illustrated. Price 3s. 6d. post paid.

The purpose of this book is to provide information to enable engineers to make more effective use of rubber, which is now pretty extensively applied to engineering practice. The author was formerly in charge of the Rubber Laboratories of Imperial Chemical Industries Limited, where much research and development work for the Services was conducted during World War II. In the course of this work he had many contacts with engineers, and this has enabled him to view the subject from the engineering point of view.

The engineering uses of rubber now cover a very wide field. Some of the more important are anti-vibration units; flexible couplings; belting, including transmission, conveyor, and elevator; hose; seals; and linings to resist abrasion or chemical attack. In most anti-vibration units the rubber is bonded to metal and is used in compression, shear or torsion, or in a combination of these stresses. Rubber in compression is an excellent shock absorber, as its non-linear upper stress-strain curve indicates that it opposes the motion with increasing force. In addition, it has no sudden stop like a steel spring. It is unique as a storer of energy, and can be used in damp positions without corroding. The greatest flexibility of rubber over a range of deflections is, however, obtained by using it in shear. The stress-strain curve in shear is practically linear up to the maximum stress used in components. This more or less ideal use of rubber was only made possible by the development of bonds to metal stronger than the rubber itself. Rubber is used in torsion in the torsion bush in which the rubber is disposed between two concentric metal cylinders or sleeves. The testing of rubber, including the testing of rubber-to-metal bonded components, is fully discussed in the final chapter.

"Dighe e Traverse," Projetto e Construzione (Dams and Barrages, their Design and Construction), by Prof. Ing. Contessini, published by Libreria Editrice Politecnica Cesare Tamburini, Milan, 1953, 7 in. by 9½ in., 595 pp., 479 ff. Price in Italy, cloth bound, L.6,000.

In view of the extent to which Italy has developed and is still developing her water power resources, it is rather remarkable that her technical literature on dams and related structures has hitherto been restricted to a number of monographs, most of them published by the power companies concerned, and articles and papers on individual installations; in fact, the publishing of an Italian comprehensive work was long overdue. The gap has now been filled by Professor Felice Contessini's handsome volume.

The book is divided into five main sections, of which the first deals in three separate chapters with the types of minor barrages and structurees generally met with in low-head and run-of-river plants—that is, fixed weirs, sluice gates, and mobile weirs.

Gravity dams are grouped in the second section, and include not only solid gravity dams, but also buttressed, multiple-arch and flat-slab dams, as well as earthfills and rockfills. This grouping, claims the author, is justified by the fact that all these types of dams are actually gravity structures, whose design and dimensioning proceeds from a common method of calculation. Logically, the chapter on dam calculation which concludes this section (VII, p. 207), embraces all these types of dams. Typical examples of practical application are included.

Arched dams are the subject of the third section (chapter VIII, p. 229), which extends to 70 pages and comprises, in addition to generalities, a discussion of the static problem, the various methods of calculation, a note on the evolution of arched-dam design, in which foreign and Italian structures are dealt with separately, and a paragraph on arched gravity dams. This section concludes with a review and discussion of the techniques applying to the design of arched dams.

The various problems connected with dam construction are tackled in the fourth section, of which the first chapter (IX, p. 301) covers the technique of foundations in all its aspects: technical characteristics of soils and rocks, geotechnical investigations, rock foundations, grouting, foundations on weathered or soft ground. A separate paragraph deals with river diverting and cofferdams, and describes a few typical diversion works such as Genissiat on the Rhone, the Tennessee diversion at Pickwick Landing, etc. With chapter X we enter upon construction problems proper, which are dealt

with exhaustively. There are separate paragraphs on concrete, cement and their components, as well as on constructional methods, and the heightening and reinforcing of existing dams. A most valuable feature of this chapter is the paragraph devoted to the organisation and layout of the plant used in dam construction. This is followed by an account of the equipment and working of large-scale construction plants, in which the author, in condensed form, does full justice to a subject which, in previous works on dams, has not always been given the consideration it deserves. No detail of organisation (transport and access, dwelling accommodation, power and water supply, laboratory, etc.) or working (aggregate supply and preparation, concrete preparation, transport and placing of concrete, mechanical equipment, handling plant, etc.) has been forgotten. Well illustrated descriptions of installations of interesting construction plants in Italy and Switzerland are added.

The behaviour of dams is reviewed in chapter XI (p. 499), particular stress being laid on control and measuring methods; some examples of typical deformations are discussed in a separate paragraph.

The fifth and last section of the book (chapter XII, p. 527) describes and discusses the various patterns of flood-discharge arrangements. An extensive bibliography, covering Italian and foreign literature, is appended to every chapter.

Professor Contessini's work is an excellent addition to international dam literature, both as a reference work and a textbook. The book is exceptionally well produced in every respect, the illustrations, photographic and line alike, leaving nothing to be desired.

British Standards Yearbook, 1954. Published by the British Standards Institution, British Standards House, 2, Park Street, London, W.1. 500 pp. Price 12s. 6d.

This yearbook lists no fewer than 2,500 British Standards current at 31st March, 1954, and gives a brief description of the subject matter of each. A comprehensive index simplifies reference.

The usual information on the membership of the Institution's General Council, Divisional Councils and Industry Standards Committees is given, together with the names of representatives on the main Special Committees and Advisory Committees. Particulars of the work in hand of all the Industry Standards Committees are also given.

The publication forms an admirable means of keeping up-to-date with the increasing momentum of practical standardisation and simplification.

I.E.C. Standard System Voltages. Publication No. 38 (third edition). International Electrotechnical Commission, 39, route de Malagnou, Geneva, Switzerland. Price 3 Sw. fr. net.

The third edition of this I.E.C. publication contains tables of standard voltages as follows:—

Table I: Standard values of the nominal voltages for single-phase systems having a voltage between 100 and 1,000 V inclusive.

Table II: Standard values of the nominal voltages for three-phase systems having a voltage between 100 and 1,000 V inclusive.

Table III: Standard values of the nominal voltages for direct-current traction systems.

Table IV: Standard values of the highest voltages, and usual values of the nominal voltages, for three-phase systems having a nominal voltage between 1 kV and 46 kV inclusive. This table contains two series, the first based upon current practice in Europe and the second upon current practice in the U.S.A. and Canada.

Table V: Standard values of the highest voltages, and the usual values of nominal voltages for three-phase systems having a nominal voltage of 60 kV and above.

Flow Problems with Respect to Intakes and Tunnels in Swedish Hydro-Electric Plants. By Lennart Rahm. Bulletin No. 36 of the Institution of Hydraulics. Royal Institute of Technology, Stockholm, Sweden. 1953. 214 pp. Illus. Price 18 Sw. kr.

This publication, in English, outlines the problems of water flow in the upstream region of a tunnel intake, and in the intake itself, in hydro-electric schemes involving underground power stations. The investigation was also extended to the discharge tunnels, which, in Sweden, are often of considerable length. This research was chiefly undertaken to determine what energy losses were incurred at these points, and if possible to devise some means of reducing them.

It has been found, for example, that vortices frequently arise at intakes, and incur both damage and losses of energy. The state existing at a number of plants was studied, and although it is possible to formulate general principles as to the cause of vortices, the conditions of flow are usually so very complicated that it has not been found possible to subject them to strict mathematical analysis. In consequence, it is concluded that a much more reliable method is to use models so that different alternatives can be compared and the design modified until the most satisfactory solution is obtained. It is recommended that these tests should be carried out at an early stage in the designing of the plant.

A further study is concerned with the types of flow which are encountered when water is discharged through a vertical pipe fitted as an overflow to a reservoir. The discharge capacity of such an installation is necessarily a matter of importance. Here again the model-testing method was adopted, and the conditions determined to secure full-flow discharge. These results were then applied to analyse and explain the observed variations in the discharge capacity of the diversion tunnel at Harsprånget, which had a vertical inlet.

A final study deals with the friction losses when turbulence is fully developed in rough conduits, with special reference to unlined rock tunnels in Sweden. The losses in head, at various discharge rates, have been determined in nine rock tunnels connected with Swedish hydro-electric stations at which measurements could be made at extremely high Renolds numbers. For unlined tunnels at high Renolds numbers a simple relationship is shown to exist between the relative area variation and the coefficients of friction in Chezy's formula. This relation shows that the coefficient of friction rapidly increases as the relative variation in the cross-sectional area increases, thus making it possible to fix this coefficient and to calculate the friction losses before the tunnel is filled with water. As a corollary, it is also possible to check these values during the actual tunnelling operations, and to adjust the cross-sectional area of the tunnel and the surge chamber, and so obtain the most economical design as the work proceeds.

Correspondence

New Graphs Aid Open-Channel Calculations

To the Editor of WATER POWER

Since writing the article "New Graphs Aid Open-Channel Calculations," which appeared in the September issue of your journal, a more direct solution to Example 4 has presented itself. This type of problem in which the quantity appears as the unknown is the most difficult to solve so that any simplification of the solution is desirable.

From equation (4a) we have

$$S_f = S_o - dy/dx\ (1 - S_r)$$

Assuming initially that the term $1 - S_r$ is unity, we get

$$S_f = 0{\cdot}0005 + 0{\cdot}00073 = 0{\cdot}00123$$

(dy/dx being negative)

From the nomograph, knowing that for this first station

$K = 6200$, we obtain $Q = 7300$ cusecs.

Now testing for S_r in the nomograph, for $Q = 7300$ cusecs and $M = 3100$, we obtain $S_r = 0{\cdot}2$

Therefore

$$S_f = 0{\cdot}0005 + 0{\cdot}00073\ (0{\cdot}8) = 0{\cdot}00109$$

Whence

$Q = 7000$ cusecs

Again this solution should be checked for each station.

In example 3 of the same paper a misreading of the K function as 660 instead of 695 and of y_o as 6·3 instead of 5·8 can be utilised so that the table remains correct if the stage is taken to within twelve inches of the normal depth

i.e. $y_1 = 5{\cdot}8 + 1{\cdot}0 = 6{\cdot}8$ feet.

R. SILVESTER.
University of Western Australia.

Asian Water Resources Conference

The report* has now been issued of the Regional Technical Conference on Water Resources Development, held recently in Tokyo under the auspices of the United Nations Economic Commission for Asia and the Far East and the U.N. Technical Assistance Administration.

The Conference re-emphasised the importance of unified river basin development, giving full consideration to all pertinent aspects, including flood control, irrigation, power generation, soil conservation, public health and recreation, as well as conservation of fish and wild life.

The Conference noted that so far emphasis had generally been laid on matters of engineering feasibility and the financial aspects of projects. It concluded that Governments should also carry out an analysis of economic justification, taking into consideration where appropriate indirect benefits which might not be susceptible of precise evaluation. Experience has shown that in connection with the river-basin development schemes, which bulk large in national budgets, analysis could not be limited to strictly financial returns.

The Conference agreed that hydrologic records, which should cover as long a period as possible, were not only fundamental to the planning of water-resources development projects, but also to the design of structures and hydraulic model investigations. The inadequacy in number of hydrologic stations and the unevenness of their distribution and the consequent difficulties encountered in planning and design were clearly illustrated with examples by many delegates. The main reason for this, as pointed out by many delegates, was the lack of resources, financial and otherwise.

In the course of the discussion on the selection of types of hydraulic structures, delegates presented considerable information relating to their experience in connection with the selection of types of structures in water-resources development. Concerning the question whether medium-size dams should be of concrete or stone masonry, it was pointed out that concrete construction was disadvantageous as it necessitated the use of expensive machinery requiring foreign exchange. On the other hand, construction of stone masonry was much slower than concrete, thereby delaying the enjoyment of project benefits and adding to the establishment and interest costs. A number of suggestions were made urging the utilisation of local labour and materials in preference to that of imported machinery and materials, other things being equal. However, the view was also expressed that more rather than less ways of utilising machinery should be found, though the countries that have to import machinery should proceed with caution.

In the discussion on the sediment problem, emphasis was placed on the planning of sediment control works, their technical features and economic justification. The economic significance of the increasingly effective control of erosion and siltation was brought out by a number of speakers, as well as by the papers presented. It was also pointed out that the potential economic savings and social gains were exceedingly large. The problems ranged from the largest river basin down to very small river basins, and from headwaters to the lower reaches of streams and canals. The role of forests in soil conservation and the effectiveness of check dams in stream control were fully discussed. Full-scale as well as experimental works were in progress in many countries, from which basic data could be obtained for use in planning and designing erosion control works.

No specific conclusions were reached on recent progress in multiple-purpose projects and no specific recommendations were made, but it was generally agreed that such exchange of information should be continued. The thirteen papers presented to the Conference were noted, and additional material was submitted during the discussion.

Discussion on organisation for water-resources development proceeded along two general lines: organisation for control of construction by the State or by contractors, and general administration of multiple-purpose projects and river-basin development.

The proposed standards on methods, records and terminology of hydrologic measurement prepared by the Bureau of Flood Control and Water Resources Development of the Commission's Secretariat were considered by a Committee on Hydrologic Standards, and a summary and recommendations issued.

* "Report of the Regional Technical Conference on Water-Resources Development," issued by the United Nations Economic Commission for Asia and the Far East, Bangkok, Thailand. 42 pp. memeograph E/CN.11/391. 30 June 1954.

High-Head Penstocks in the Rockies - II

Design and construction features of the penstocks for the Pole Hill and Flatiron power plants of the Colorado-Big Thompson project.

By PETER J. BIER

Design and Specifications

The penstocks were designed for the maximum head anticipated under normal operating conditions. A minimum turbine gate closure of five seconds was assumed as a basis of speed and pressure-rise computations. At Pole Hill the maximum static head on the penstock is 845 ft. and at the Flatiron power plant 1,132 ft. Each turbine was equipped with a pressure regulator or relief valve, having a discharge capacity of 100 per cent. of the turbine capacity at full gate. The maximum water hammer, therefore, will normally occur during gate opening with oncoming load for which condition relief valves are not effective. According to Allievi's counterblow theory, a sudden gate opening under a head in excess of 100 m. is followed by a pressure oscillation which may result in a super-pressure (counterblow) of 23 per cent. of the static head. This raises the maximum operating head to 1,040 ft. for Pole Hill and to 1,392 ft. for the Flatiron penstocks. These computed heads were used in the design.

Steel of firebox quality was used throughout the penstocks. For the upstream or low-head portions, comprising about 1,500 ft. of length for Pole Hill and about 2,720 ft. for each Flatiron penstock, a low and intermediate tensile-strength carbon steel, ASTM Designation A285, Grade C was used. This type of steel is required to have a minimum tensile strength of 55,000 lb. per sq. in., a minimum yield point of 30,000 lb. per sq. in. and a minimum elongation, in 8 in., of $\frac{1{,}550{,}000}{\text{tensile strength}}$. The maximum carbon content is specified at 0·20 per cent. for plates $\frac{3}{4}$ in. and under; and 0·22 per cent. for plates over $\frac{3}{4}$ in. thickness. A maximum manganese content of 0·80 per cent. is also demanded. For the downstream or high-head portions of all penstocks, a high-strength carbon-silicon steel, ASTM Designation A212, Grade B, was used. Because of the necessity to field weld the two halves together, all butt straps were made from A285 Grade C steel. The ASTM standard specifications prescribe the following properties for A212 Grade B steel:

Minimum tensile strength: 70,000 lb. per sq. in.

Minimum yield point: 35,000 lb. per sq. in.

Minimum elongation, in 8 inches $\frac{1{,}550{,}000}{\text{tensile strength}}$

Maximum carbon content: 0·31 per cent. for plates 1 in. and under, and 0·33 per cent. for plates over 1 in. to 2 in. inclusive.

Maximum manganese content: 0·90 per cent.

The field girth joints of the penstock sections made from A285 steel were required to be welded and those made from A212 steel to be riveted. As an alternative, field girth joints of sleeve-type couplings were permitted throughout each penstock. An exception was made in the buried downstream sections of each penstock, where one sleeve-type coupling was to be used as a make-up joint in each tangent section, except in the end tangent of Flatiron penstock No. 1 This section was required to have riveted field girth joints to carry the bulkhead load acting on the butterfly valve when closed. A concrete anchor was provided in the downstream end section of penstock No. 2 to carry this bulkhead load.

The design was based on the provision that all penstock sections fabricated from A212 steel, and from A285 steel when over 1 in. thickness, would be thermally stress relieved after welding. Thermal stress relieving was also specified for the sleeves and packing glands of the expansion joints to facilitate machining and secure dimensional stability. Preheating prior to welding was required for plates of A212 steel over $\frac{5}{8}$ in. thickness. The stress-relieving requirement was in conformance with the API-ASME code for fusion-welded unfired pressure vessels, 1943 edition, which was followed as far as possible in the design. In computing the plate thicknesses, a design stress of 13,750 lb. per sq. in. was used for the A285 Grade C steel, and 17,500 lb. per sq. in. for the A212 Grade B steel. The conventional thin-shell formula of $t = \frac{rp}{se}$ was used, assuming a joint efficiency e of 95 per cent. for stress-relieved sections and 90 per cent. for sections not stress relieved. These efficiences were also based on the use of double-welded butt joints fully radiographed.

The minimum plate thickness of $\frac{5}{16}$ in. at the upstream ends of the penstocks was computed from the formula t minimum $= \frac{d+20}{400}$, where d = diameter of penstock in inches. Plate thicknesses for the downstream sections of the penstocks based on A212 Grade B steel, were computed for a range of $\frac{3}{4}$ in. to $1\frac{5}{16}$ in. Based on A285 Grade C steel throughout, the thicknesses varied from $\frac{5}{16}$ to $1\frac{11}{16}$ in. For the closing sections at the downstream ends of the penstocks connecting them to the turbines, for which A285 Grade C steel was specified, a plate thickness of $1\frac{1}{8}$ in. was used for Pole Hill and $1\frac{11}{16}$ in. for Flatiron. All closing sections were to be fabricated from A285 Grade C steel to avoid the necessity of field welding A212 steel without stress relieving.

Supports were spaced at 60 ft. centres, measured along the slope of the pipeline, except at expansion joints where shorter spans were used. Over the points of support, the plate thickness was increased by $\frac{1}{8}$ in. above that of the adjoining sections, for a length of 4 ft., to reduce secondary bending stresses. The supports were further stiffened by double ring girders

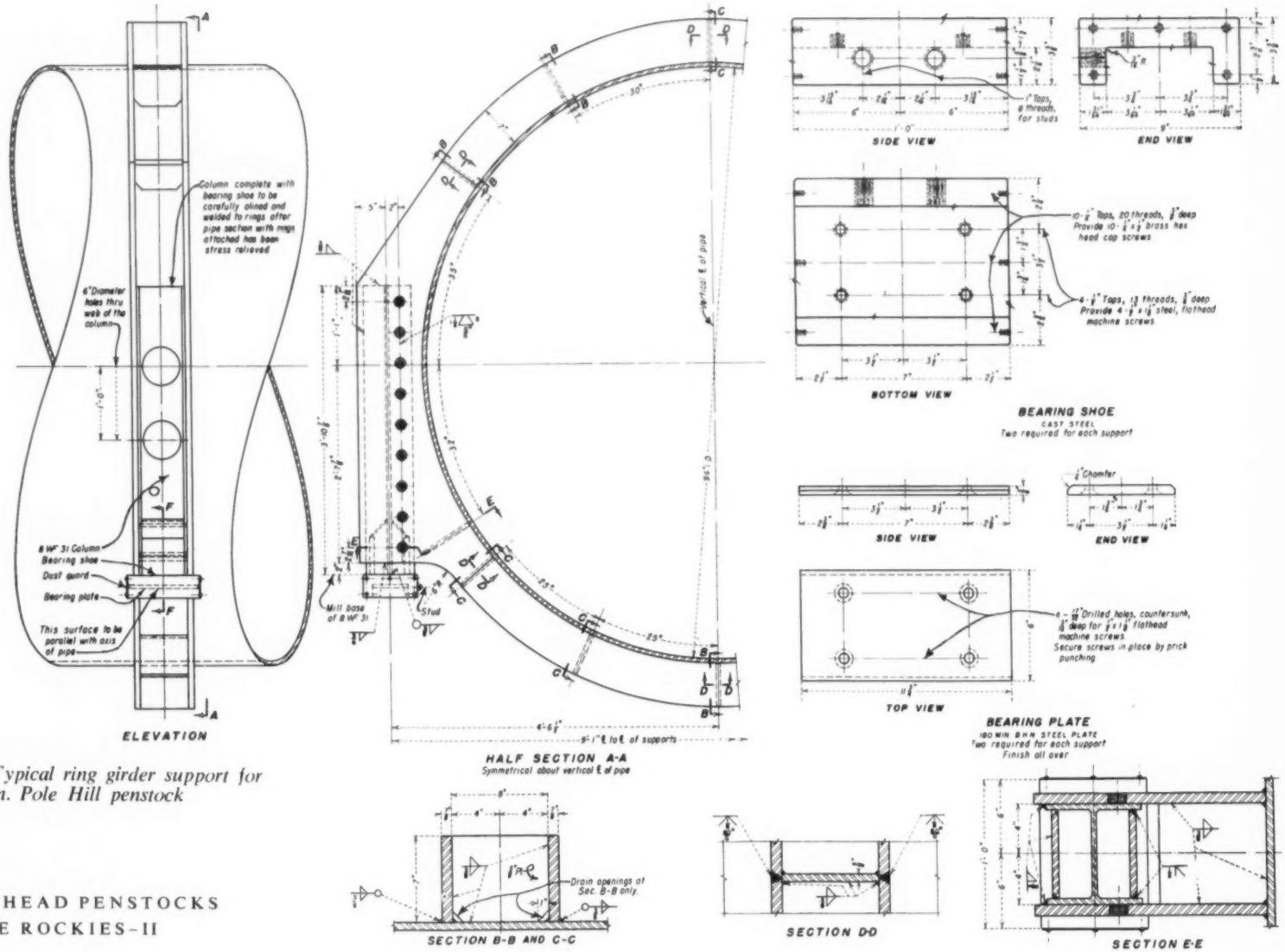

ELEVATION
Column complete with bearing shoe to be carefully aligned and welded to rings after pipe section with rings attached has been stress relieved
6" Diameter holes thru web of the column
8 WF 31 Column
Bearing shoe
Dust guard
Bearing plate
This surface to be parallel with axis of pipe
Mill base of 8 WF 31
Stud
HALF SECTION A-A
Symmetrical about vertical ℄ of pipe
SECTION B-B AND C-C
Drain openings at Sec. B-B only.
SECTION D-D
SECTION E-E
SIDE VIEW
END VIEW
BOTTOM VIEW
BEARING SHOE
CAST STEEL
Two required for each support
TOP VIEW
BEARING PLATE
Two required for each support
Finish all over

Fig. 9. Typical ring girder support for the 96 in. Pole Hill penstock

welded to the shell. Two columns attached to the girders carry the load to the rockers and support piers. Fig. 9 shows a typical ring girder support for the 96 in. Pole Hill penstock. The girder consists of two $\frac{7}{8}$ in. by 7 in. rings welded to the pipe shell and tied together with a series of diaphragm plates welded between. The support columns consist of 8 in. wide flange beams with milled ends. Machined cast-steel bearing shoes, lined with hardened-steel bearing plates, were welded to the column bases to transmit the load to the rockers. Fig. 10 shows a typical rocker assembly for the Pole Hill penstock. The rockers carry the load to the base castings, which are grouted and anchored into the concrete piers. A steel pin with split bronze bushings and provided with Lubrite inserts, transmits the load from the rocker to the base casting. Two steel collars connect the rocker to the base casting which forms the lower half of the bearing for the pin. To prevent the rockers from losing contact with the pipe when the latter rises or the support pier settles, a steel tooth is provided to keep the rocker in alignment with the pipe.

The design also made provision for installing the rockers in accordance with the prevailing temperature. At a penstock temperature of 40°F. the rocker should be in central position, allowing equal deflections at each side to accommodate high and low-temperature limits. After this adjustment had been made in accordance with a temperature chart, the base casting was required to be grouted into the pier, while the rocker assembly was being held in position by means of erection fixtures until the grout had set sufficiently to carry the load. In the meantime the penstock load was to be carried on temporary timber supports.

All bends were designed with long radii and small deflections between segments to reduce hydraulic losses. Reducing bends were used at points of diameter changes. Where the profile required deflections in both vertical and horizontal planes combined bends were provided. A typical combined bend for the Flatiron penstocks is shown in Fig. 11.

The expansion joints were designed to permit temperature movements due to expansion and contraction in the penstock tangents between anchors. They also served as make-up joints to permit adjustments in tangent lengths during installation. A typical expansion joint for the 96 in. diameter Pole Hill penstock is shown in Fig. 12. It consists of an inner and an outer sleeve, a packing gland, a seat ring for the packing, and the packing. The inner sleeve is made of heavier plate than that used in the adjoining penstock sections. It is protected with chromium cladding on the outside to prevent corrosion. Outer sleeves are made from plate of sufficient thickness to resist deformation and permit tapping for the gland bolts. The packing gland is provided with a number of stud bolts used for tightening the packing, which consists of six rings of 1 in. square lubricated long-fibre braided flax. All expansion joints were required to be fabricated within close tolerances to assure watertightness, and consequently machining was specified for some surfaces of the packing gland and the outer sleeves. The inner sleeve was required to be butt welded to the adjoining upstream penstock section, and the outer sleeve to be lap welded to the downstream section, using a filler plate between sleeve and penstock, as shown in Fig. 12, to accommodate the larger diameter.

The specifications required that the penstock sections should be fabricated within certain tolerances in length and diameter, and that the inner surfaces of plates and penstock courses should not have offsets greater than $\frac{1}{16}$ in. Courses were to be made from single plates of 120 in. net width. A tolerance of plus or minus $\frac{3}{8}$ in. between centres of support piers was specified. Penstock sections to be connected with sleeve-type couplings were to be not more than 1/32 in. smaller nor more than 3/32 in. larger than the nominal diameter. The outside surfaces of such penstock sections were required to be smooth, and free from irregularities for at least 10 in. from the

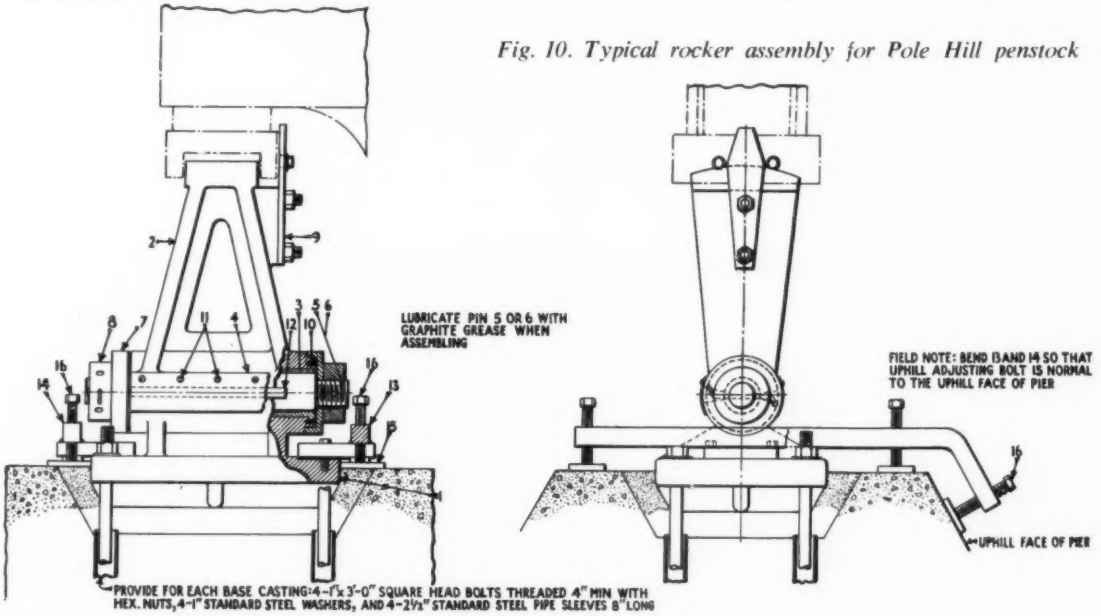

LUBRICATE PIN 5 OR 6 WITH GRAPHITE GREASE WHEN ASSEMBLING
FIELD NOTE: BEND 13 AND 14 SO THAT UPHILL ADJUSTING BOLT IS NORMAL TO THE UPHILL FACE OF PIER
UPHILL FACE OF PIER
PROVIDE FOR EACH BASE CASTING: 4-1"x 3'-0" SQUARE HEAD BOLTS THREADED 4" MIN WITH HEX. NUTS, 4-1" STANDARD STEEL WASHERS, AND 4-2½" STANDARD STEEL PIPE SLEEVES 8" LONG

Fig. 10. Typical rocker assembly for Pole Hill penstock

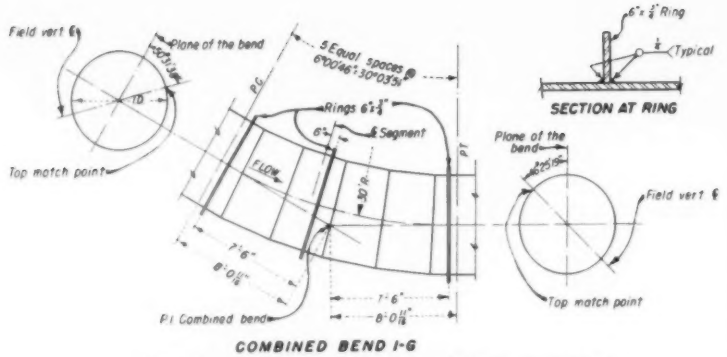


Fig. 11. Combined bend for Flatiron penstocks

ends of the sections, to provide watertight joints with the rubber gaskets of the coupling.

Welding wherever possible was to be performed on automatic machines. All intermediate layers of deposited weld metal were to be cleaned of slag and other deposits before the next layer was deposited. Tack welds were to be chipped out before the final welds were made, and weld reinforcement on the inside of the pipe was to be limited to a maximum of $\frac{1}{8}$ in. Defective welds were to be chipped or flame gouged to sound metal and replaced with sound welds. Flanges were required to be welded to their respective penstock courses before final machining.

Each penstock was provided with a number of 20 in. diameter manholes, for inspection and maintenance. The manholes were spaced about 500 ft. apart, and were placed either on top or at the bottom quarter point of the penstock. The manholes on top of the penstocks are for ventilation purposes during painting. Hinged steel covers with rubber gaskets are used for closure. Drain connections of 6 in. diameter, controlled with plug valves, are located near the downstream end of each penstock, to facilitate draining the line when necessary, for maintenance and inspection. Piezometer and pitometer connections were provided for use during the initial turbine acceptance tests. Dished heads were welded near the downstream end of each penstock, upstream from the closing section, to facilitate filling in preparation for the leakage test made after installation. Separate filling lines were not provided for the penstocks, as filling can be accomplished by partial opening of the fixed-wheel gate at Pole Hill, and of the butterfly valves in the Flatiron intake structure.

All anchors were designed for the combination of forces acting on them. These include temperature forces, hydrostatic and dynamic forces, also forces on reducers and gravity forces due to the weight of the pipe and water. As these combined forces must be resisted by the anchor its volume was so proportioned that the weight of concrete combined with the weight of the section of pipe and water supported on the anchor would provide a safe frictional resistance against sliding between the concrete and the underlying soil. To increase the resistance against sliding, anchors were set on rock foundations where possible, for which condition a sliding coefficient of 0·65 could be used. Where rock was not within reach, anchors were placed on existing soil, using sliding coefficients from 0·35 to 0·50 depending on its quality. Fig. 13 shows a typical concrete anchor designed for the Pole Hill penstock. This anchor contains 208 cu. yards of concrete and 12,600 lb. of reinforcement steel, or about 60 lb. per cu. yard of concrete. This anchor is founded on solid rock, and has been proportioned to resist the computed forces. For a safe anchor design, the resultant of the combined forces should intersect the base in its middle third. Stiffener rings, encased in the anchors, were specified to fix the pipe in place in the concrete.

The support piers were designed to resist safely the frictional forces due to temperature movement of the pipe, and to carry the weight of the pipe and contained water. With the use of rockers, the frictional force tending to overturn the pier is reduced from about 60 per cent. (when pipe is supported directly on a concrete cradle) to about 10 per cent. Fig. 14 shows a typical concrete pier design for the Flatiron penstocks. Piers were reinforced for the bending

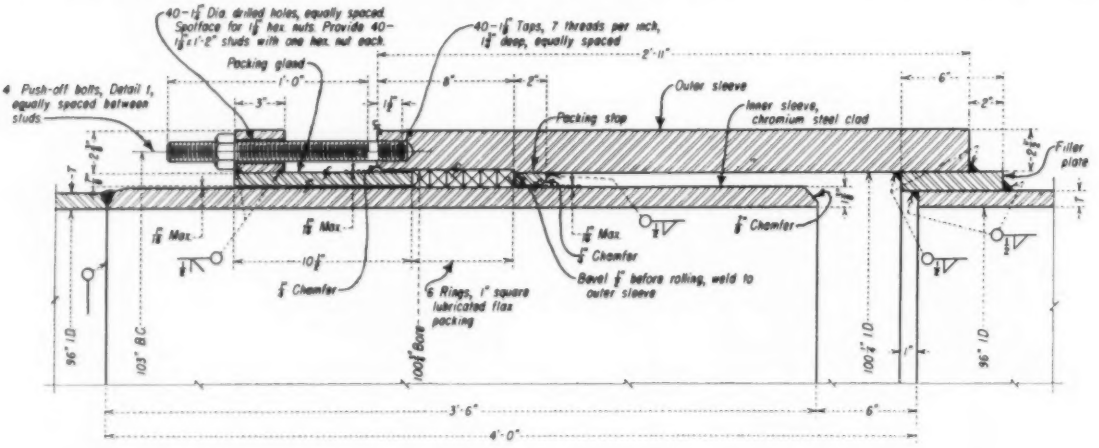


Fig. 12. Typical expansion joint, showing filler plate between sleeve and penstock

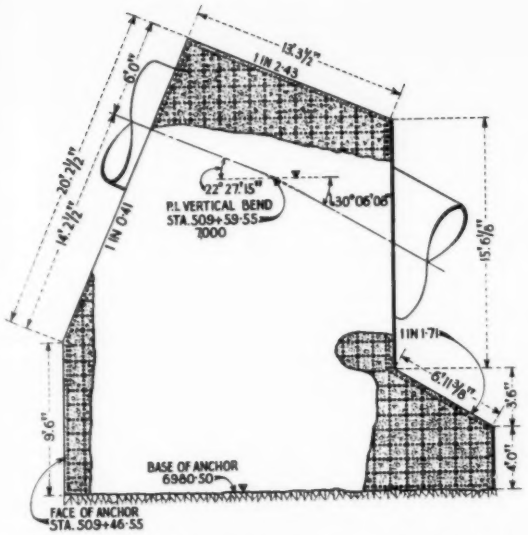


Fig. 13. Typical concrete anchor designed for the Pole Hill penstock

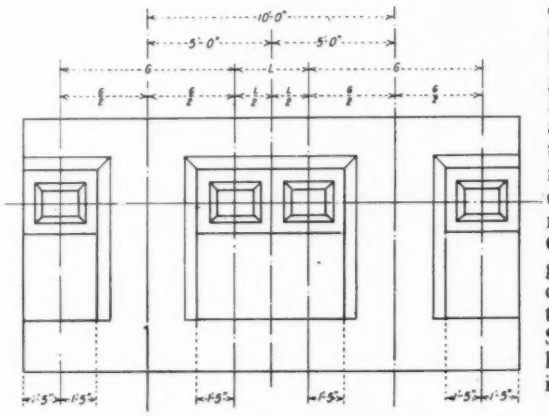
moment produced by the frictional forces and for the shear forces in the base. All piers were extended sufficiently above the ground surface to provide a clearance of 2 ft. under the exposed pipe. The anchors and piers for the two Flatiron penstocks were designed as double units to accommodate both pipes.

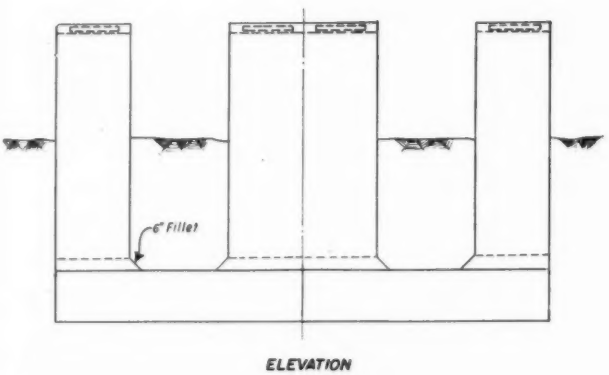
The specifications required that each penstock section, after welding, radiographing, and stress relieving, should be subjected to a hydrostatic pressure test to develop a hoop tension of 20,000 lb. per sq. in. in the A285 steel and 26,000 lb. per sq. in. in the A212 steel. A leakage test for each installed penstock was also called for, to be performed under static head only, the primary purpose being to determine the watertightness of the riveted field girth joints.

Construction Alternatives and Bids

The specifications required the use of A285 Grade C steel for penstock sections with the plates proportioned in accordance with the minimum thickness formula, but made it optional to use either A285 Grade C or A212 Grade B steel for sections with plates proportioned for the operating head. For various A285 plates it was also optional to use either welded, riveted, or sleeve-coupled field girth joints. For A212 steel, all field girth joints were to be riveted or sleeve-coupled. Designs were prepared for both types of construction to give bidders an opportunity to quote on either type. Fig. 15 shows a riveted field girth joint for the A212 Grade B steel used in the Flatiron penstocks. Double-riveted joints with $1\frac{1}{8}$ in. and $1\frac{1}{4}$ in. rivets were specified for plates from $\frac{3}{4}$ in. to $\frac{15}{16}$ in. thickness and triple-riveted joints with $1\frac{1}{4}$ in. rivets for the remainder of the plate thicknesses. The design of the riveted joints was based on the use of rivets in accordance with ASTM Designation A31, Grade B. The reason for not permitting welded field girth joints in the A212 steel and in the A285 steel over 1 in. thickness was the inability to comply with the Code's requirement to stress relieve these welds. Such stress relieving would have to be performed locally for each field joint, which is considered to be impracticable.

In addition to the field girth joints, the principal difference between the welded-riveted and sleeve-coupled alternatives was in the number and size of





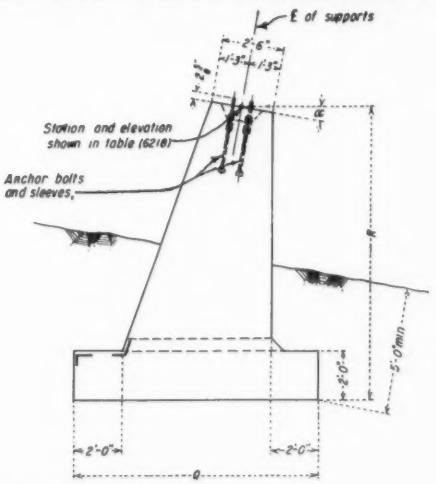


Fig. 14. Typical concrete pier for the Flatiron penstocks

the piers and anchors, and in the use of expansion joints for the welded-riveted type. When using sleeve-type couplings (Dresser or equal), the conventional-type expansion joints can be dispensed with, as the couplings will function as expansion joints for each separate pipe section. They absorb the usual temperature movements, as expansion and contraction, by shear deflection in the rubber gaskets. Sleeve-coupled penstocks, however, require different supports and anchors from the welded and riveted type. To ascertain the thrust acting on an anchor of a sleeve-coupled penstock, it is customary to assume arbitrarily different temperature forces at each side of the anchor. Theoretically, these forces are of equal magnitude, but may become unbalanced for various reasons, which should be taken care of in the design.

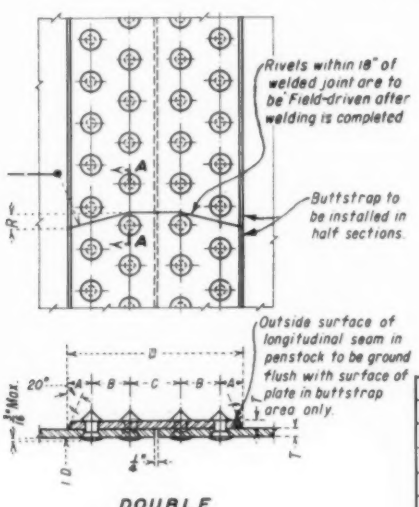
The specifications also permitted alternative concrete constructions for the upstream portions of each penstock to a point where the design head reached 390 ft. for Pole Hill and 420 ft. for Flatiron. Precast concrete pipes with rubber-gasketed joints were proposed as an alternative to the steel penstocks, and bids were invited accordingly. The concrete pipe was required to be installed in a trench, supported on compacted bedding and backfilled with earth.

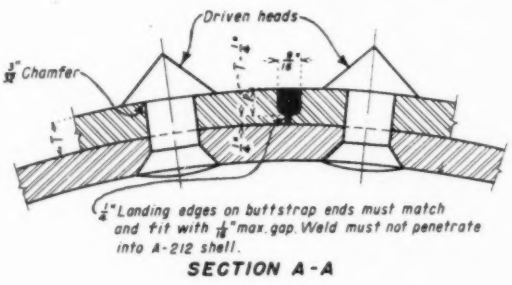
Bids were called for in June 1951. They included fabrication and installation, complete with all earth and concrete work. Two bids were received for the all-steel penstock construction and two for the steel penstock with concrete at the upstream end. The specifications required the use of an evaluation factor for alternative portions of the penstocks. A factor of 1·00 was used for the concrete and 1·04 for the steel-pipe alternative, the latter value being due to the higher maintenance cost assumed for the steel penstock. Although the bids received for the steel penstock were thus increased by 4 per cent. they were still considerably lower than the bids received for the corresponding portions of concrete penstock.

As the low bid at $4,284,512 was acceptable the contract was awarded to the Southwest Welding and Manufacturing Company of Alhambra, California. The contractor's bid was based on the use of A285 Grade C steel for the upstream portions from $\frac{5}{16}$ to $1\frac{3}{16}$ in. thickness, with welded field girth joints, and on the use of A212 Grade B steel for the downstream portions from $\frac{3}{4}$ to $1\frac{3}{16}$ in. thickness, with riveted field girth joints as described previously. For the buried sections of the penstocks at the downstream end, the contractor proposed to use Dresser couplings except in the end tangent of Flatiron penstock No. 1, which was required to be riveted.

The estimated weight of the Pole Hill penstock was 1,043 tons, and the Flatiron penstocks 5,216 tons, or a total of 6,259 tons for the entire contract.

(To be continued)


Rivets within 18" of welded joint are to be Field-driven after welding is completed
Buttstrap to be installed in half sections.
Outside surface of longitudinal seam in penstock to be ground flush with surface of plate in buttstrap area only.
20°
DOUBLE


Driven heads
Chamfer
Landing edges on buttstrap ends must match and fit with max. gap. Weld must not penetrate into A-212 shell.
SECTION A-A

72"	$\frac{13}{16}$"	Double riveted	$1\frac{1}{8}$"	64	$1\frac{3}{4}$"	$2\frac{5}{8}$"	$3\frac{3}{8}$"	$12\frac{1}{2}$"	$3\frac{1}{32}$"
72"	$\frac{7}{8}$"	" "	$1\frac{1}{4}$"	60	2"	$2\frac{7}{8}$"	$4\frac{1}{4}$"	14"	2"
72"	$\frac{15}{16}$"	" "	$1\frac{1}{4}$"	60	2"	$2\frac{7}{8}$"	$4\frac{1}{4}$"	14"	$3\frac{11}{32}$"
72"	1"	Triple riveted	$1\frac{1}{4}$"	48	2"	$3\frac{1}{2}$"	$4\frac{1}{4}$"	$22\frac{1}{4}$"	$1\frac{7}{8}$"
72"	$1\frac{1}{16}$"	" "	$1\frac{1}{4}$"	48	2"	$3\frac{1}{2}$"	$4\frac{1}{4}$"	$22\frac{1}{4}$"	$1\frac{1}{4}$"
72"	$1\frac{1}{8}$"	" "	$1\frac{1}{4}$"	48	2"	$3\frac{1}{2}$"	$4\frac{1}{4}$"	$22\frac{1}{4}$"	$1\frac{3}{4}$"
72"	$1\frac{3}{16}$"	" "	$1\frac{1}{4}$"	60	2"	$2\frac{7}{8}$"	$4\frac{1}{4}$"	$19\frac{3}{4}$"	$2\frac{1}{8}$"
72"	$1\frac{1}{4}$"	" "	$1\frac{1}{4}$"	60	2"	$2\frac{7}{8}$"	$4\frac{1}{4}$"	$19\frac{3}{4}$"	$1\frac{7}{8}$"
$71\frac{3}{8}$"	$1\frac{5}{16}$"	" "	$1\frac{1}{4}$"	60	2"	$2\frac{7}{8}$"	$4\frac{1}{4}$"	$19\frac{3}{4}$"	$2\frac{13}{16}$"

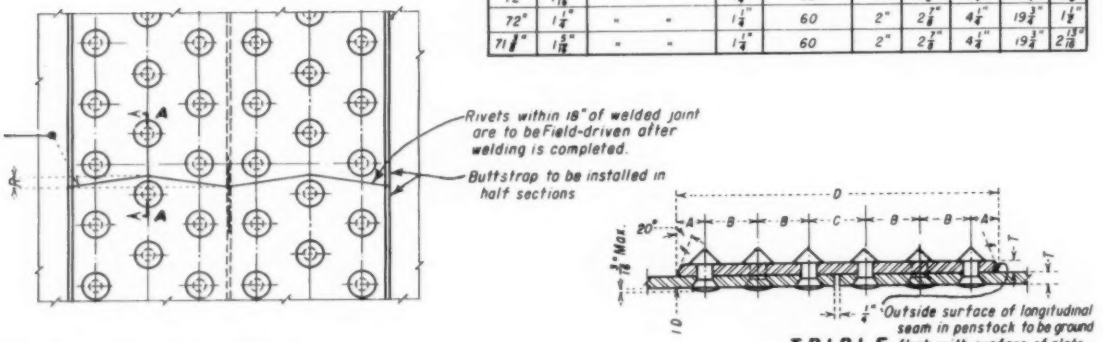

Rivets within 18" of welded joint are to be Field-driven after welding is completed.
Buttstrap to be installed in half sections
20°
Outside surface of longitudinal seam in penstock to be ground flush with surface of plate in buttstrap area only
TRIPLE

Fig. 15. Riveted field girth joint

Fig. 1. Pallivasal power station in background with pumping station and connecting channel in foreground

The Sengulam Hydro-Electric Project

A brief description is given by R. Dorai Rajan of a development in Travancore-Cochin State

THE Sengulam hydro-electric project utilises the tail waters of the Pallivasal station and a fall in the Mudirapuzha river of nearly 1,200 ft. between that power house and the Sengulam station at Vellathuval, on the North Travancore Hills. There are possibilities of using the same water for power development lower down the valley. The installed capacity of the Sengulam plant will be 48,000 kW with a firm power of 24,000 kW.

Water for operation of the Sengulam plant is diverted just below the tail race of the Pallivasal station and is led through a channel to a small forebay, whence it is pumped to a height of about 50 ft. into a tunnel which flows into a balancing reser-

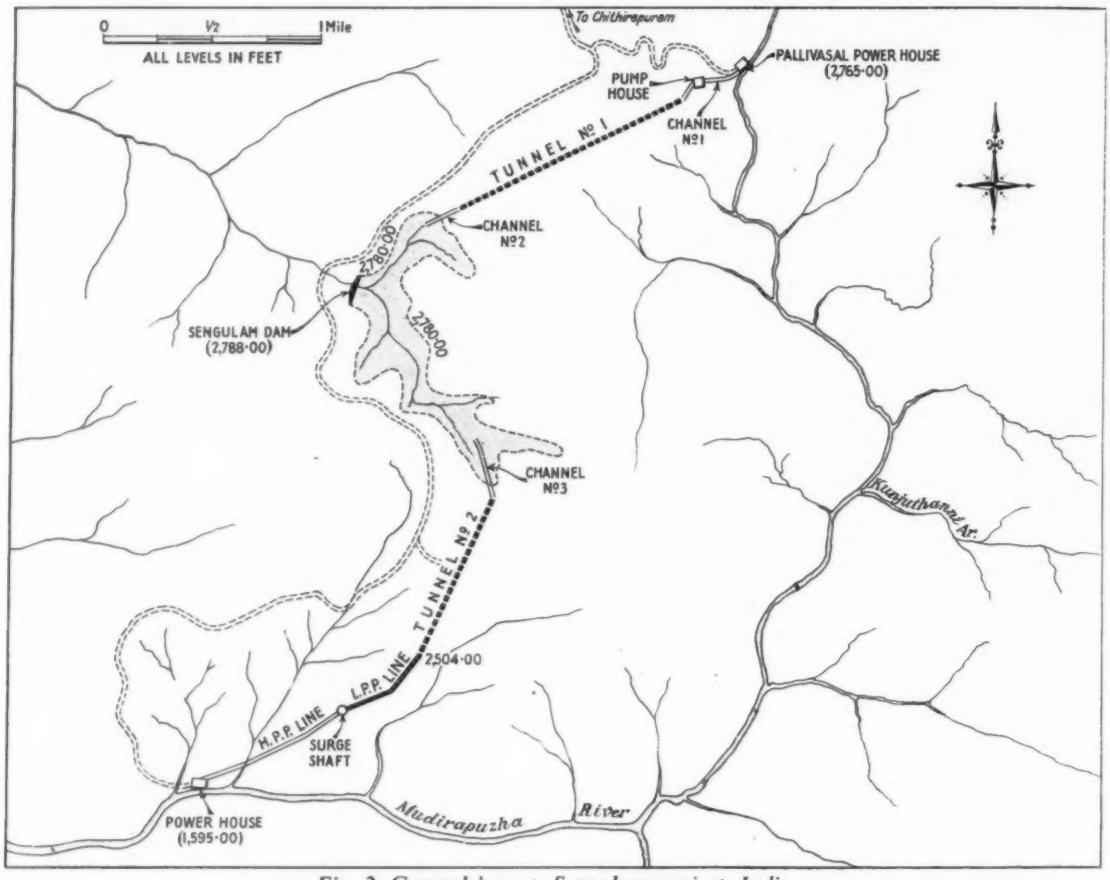


Fig. 2. General layout, Sengulam project, India

voir. A second tunnel from this point leads the water to the pipelines to the turbines.

The water channel from the Pallivasal station is of a composite nature, comprising a cut and cover channel, an open channel and an aqueduct totalling 1,510 ft. in length, with a slope of 1 in 1500. The reinforced concrete cut and cover conduit 12 ft. wide and 7 ft. high is 410 ft. long, and emerges into an open trough, 500 ft. in length and of the same height and width; it is also lined with reinforced concrete and masonry. The aqueduct, 300 ft. long, is carried on concrete piers, and leads to a pump house fore-

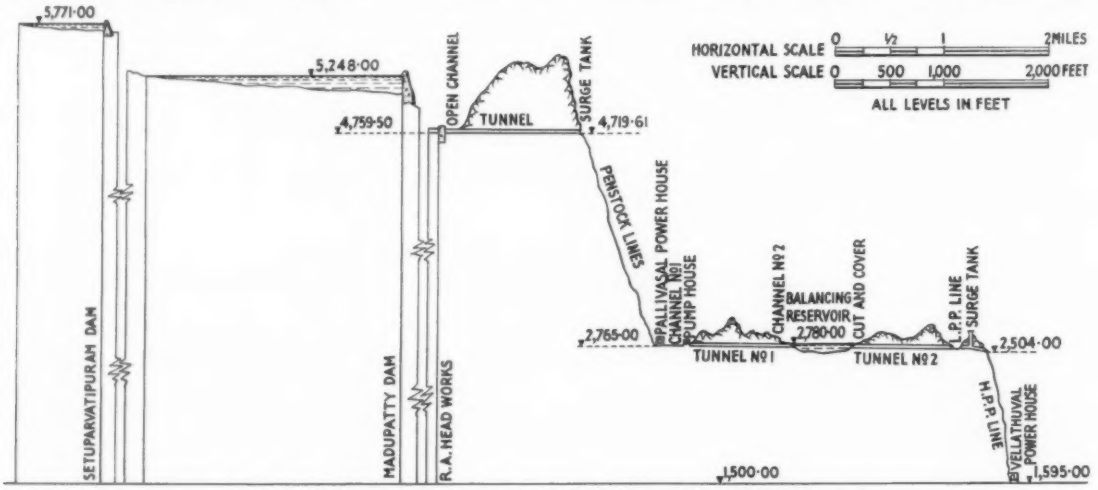


Fig. 3. Schematic diagram of Pallivasal and Sengulam schemes

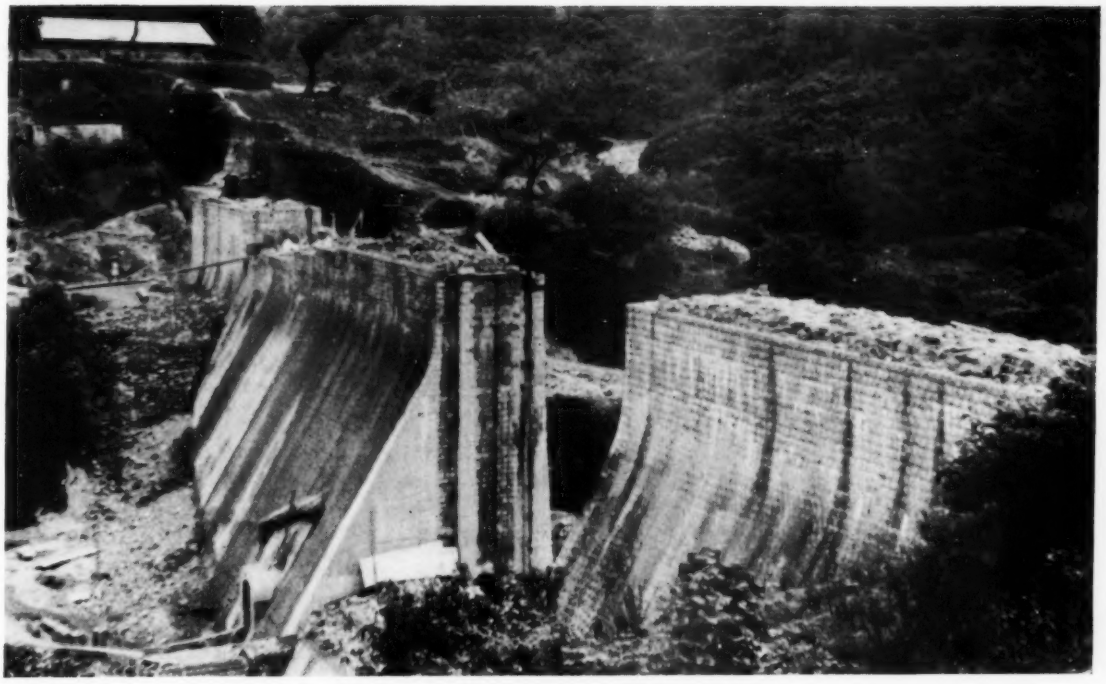


Fig. 4. Sengulam dam under construction

bay 60 ft. wide and 300 ft. long of reinforced concrete construction. The channel has a normal capacity of 400 cusec. A diversion across the Mudirapuzha river below the Pallivasal station is contemplated later on to supplement the tail water.

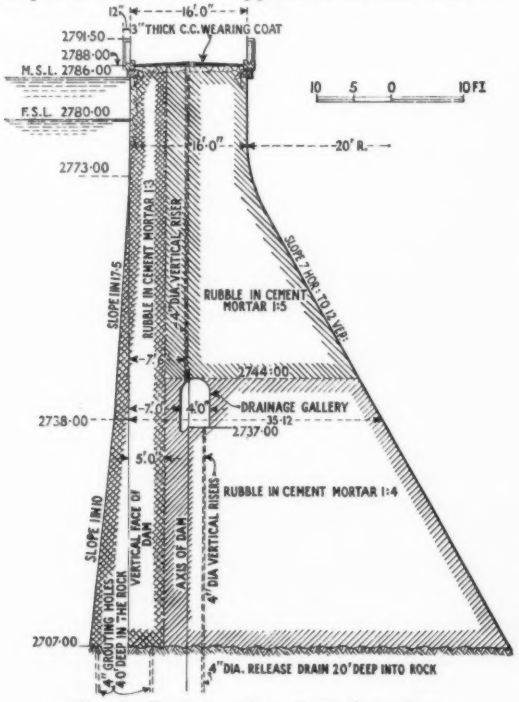


Fig. 5. Cross section, Sengulam dam

The pumphouse is a reinforced concrete framed structure 60 ft. long, 30 ft. wide and 30 ft. high above floor level, with an annexe 12½ ft. wide and 15 ft. high throughout its length, and rests on rocky foundations about 20 ft. below the ground level.

Fig. 6. Penstock track under construction

The pumps are arranged below floor level so as to be self-priming.

Water from the pump forebay is pumped to a height of 48 ft. by three vertical Sulzer pumps, type A.P.—100 S.V., each capable of discharging 150 cusec. They have a speed of 375 r.p.m. and efficiency of about 90 per cent. The drive consists of Brown-Boveri 1,050 h.p. 300 volts, three-phase, 50 cycle synchronous induction motors, of drip-proof design, directly coupled to the pumps.

The power supply for motors is derived from three Crompton 11/3 kV transformers 2,000 kVA capacity each. In addition to manual controls, the pumps are also automatically controlled in accordance with water levels in the forebay and the balancing reservoir. The pumps are connected by a short steel pipe to the inlet of the tunnel. The pump house and pumping equipment cost Rs. 14 lakhs.

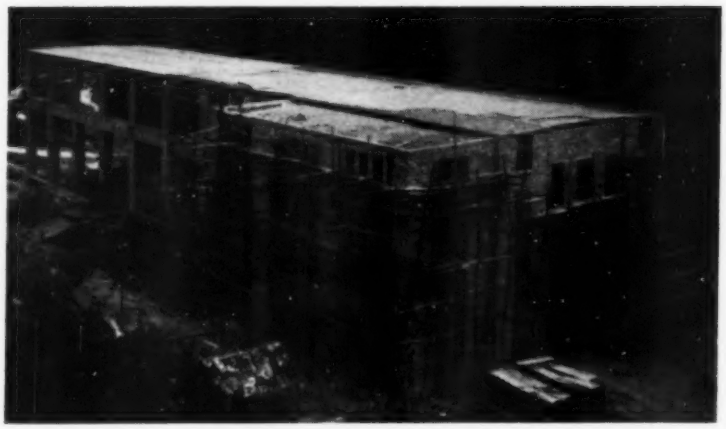


Fig. 7. Sengulam power house in course of erection

Two tunnels are driven through intervening hills; the first is 5,800 ft. long, has a finished cross-sectional area of 72 sq. ft., and empties the water into the balancing reservoir, to which reference has already been made. This reservoir is situated in a neighbouring valley and affords a storage of 24 million cu. ft., sufficient to meet the diurnal load variations of the Sengulam plant. It is formed by a masonry dam 350 ft. long (excluding 60 ft. core-walls on either side), 65 ft. above bed level and 50 ft. wide at foundations; on the top it carries a 12 ft. wide roadway. The main dam has a drainage gallery 7 ft. by 4 ft.; a central spillway 50 ft. wide and a bottom discharge sluice 2 ft. 6 in. in diameter are provided. From the balancing reservoir the water flows through a small channel to a second concrete-lined tunnel 6,770 ft. in length. This is connected to a low pressure pipeline 102 in. internal diameter and 2,650 ft. long, and held by nine concrete anchors. It ends at the cliff edge, at which point there is a surge shaft having a height of 200 ft., the diameter for the first 120 ft. being 15 ft., and the remainder 8 ft. 6 in.

Two high pressure penstocks, each 58 in. internal diameter and 3,160 ft. long, take off from the surge shaft. These two penstocks bifurcate near the power house to the 42 in. diameter distribution pipes which serve the four generating sets. Both the low pressure pipeline and high pressure penstocks were supplied by the Dortmunder Union, Burkenbow, West Germany.

A haulage track runs along the penstocks and is operated by a winch, supplied by Uskside Engineering Co. Ltd., and driven by a 75 h.p. Metrovick motor.

Just below the surge shaft there is a valve house equipped with 58 in. automatic butterfly valves for controlling the flow through the penstocks. At the power house end each distribution pipe to the unit is fitted with a hand-operated 42 in. sluice valve located outside the building.

The power house is situated on the right bank of the Mudirapuzha river, and consists of a reinforced concrete framework and panel walls of hollow cement blocks. It is 221 ft. long, 43 ft. wide and 41 ft. high. The main hall is designed to accommodate four Pelton wheel turbine generating sets, each comprising an 18,000 h.p. horizontal-shaft double-runner, four nozzle, impulse turbine directly coupled to a 12,000 kW 0·85 power factor alternator. The speed is 500 revs. per min. and generation is at 11,000 V, 3-phase, 50 cycles, this voltage being stepped up to 66,000 V by 15,000 kVA transformers directly connected to generators. Each turbine is provided with a 36 in. dia. cylindrically balanced valve installed inside the power house. The turbines, generators, switchgear and valves, which cost Rs. 76 lakhs, were all supplied by the English Electric Co. Ltd.

Such station auxiliaries as the 11 kV switchgear and the batteries are housed on the ground floor of an upstream side annexe to the house main building, while the control room is on the first floor.

The Sengulam and Pallivasal stations are connected by a 66 kV duplicate tie line about 4½ miles in length and carried on steel towers. These feeders are provided with Translay pilot-wire unit protection backed up by over-current and earth leakage protective gear. Synchronising of the incoming 66 kW feeders from the Sengulam station is effected at the Pallivasal station, the control desk at Pallivasal and the operation room at Sengulam being suitably interconnected for necessary signalling.

As more than half the installed capacity of the Sengulam plant will have to be transmitted to the southern parts of the State, work has already been started on connecting Sengulam to the Pallom and Kundara sub-stations by means of a 110 kV double-circuit A.C.S.R. line.

Mr. C. S. Padmanabhan, Executive Engineer, is in charge of the construction of the scheme, supervised by Mr. George Oommen, Chief Construction Engineer, Hydro-Electric Development, Travancore-Cochin State.

English Consulting Engineer for French Tidal Schemes. Mr. Geoffrey Kennedy has been appointed by Electricité de France as a Member of the Comité Consultatif des Usines Maremotrices which has been established for the French tidal-power schemes of the Rance and Chausey Mont Saint-Michel.

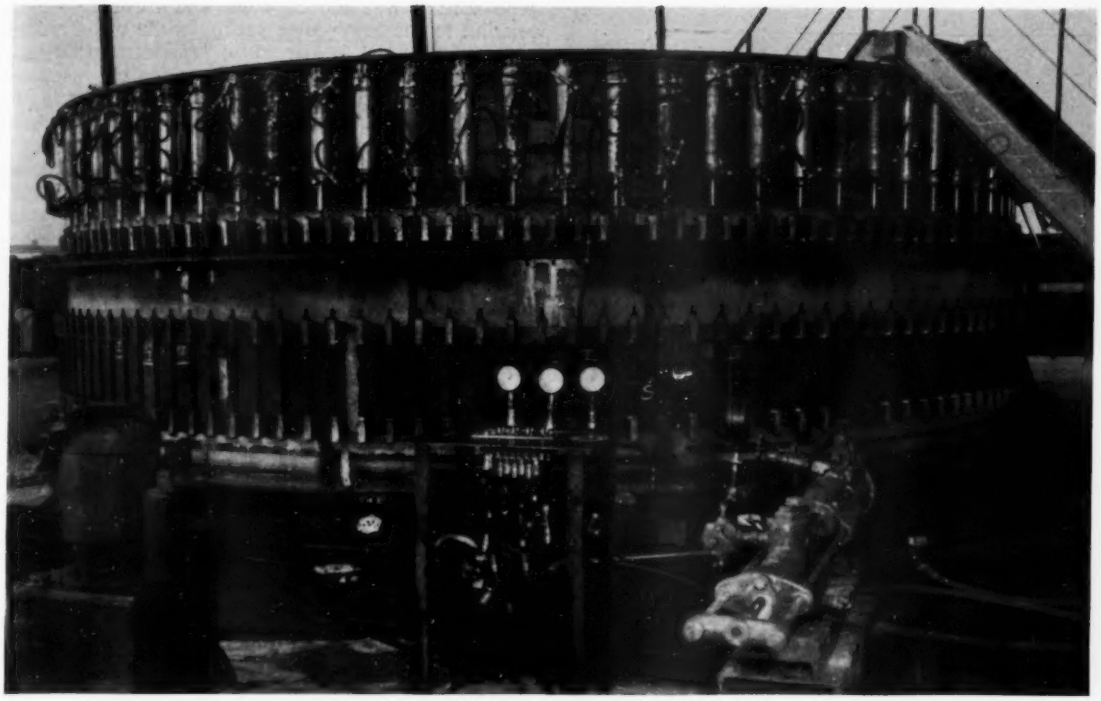


Fig. 1. One of the R. H. Baker & Company flexible penstock couplings for Garrison Dam undergoing hydraulic test

An Outstanding Flexible Coupling

When the U.S. Army's Corps of Engineers began construction of the Garrison Dam at Riverdale, North Dakota, a special problem was encountered which demanded satisfactory solution if the project was to be a complete engineering success.

Specifications called for one of the largest roll-fill earth embankments in the world stretching across the Missouri Valley for more than two miles. It rises 200 feet above the stream bed and contains 70 million cu. yards of material. As the dam sits on "Fort Union" material, the Corps of Engineers decided that the construction would be subject to settlement, rebound and other shifting. This created critical conditions for the five steel penstocks, 24 ft. in diameter, which served the turbines. In consequence, the Corps of Engineers specified that a flexible pipe coupling, or "articulating joint," be provided to make the penstocks sufficiently flexible.

R. H. Baker and Company, 2070 East Slauson Avenue, Huntingdon Park, California, solved this problem by designing a special steel coupling, consisting of a steel ring incorporating two resilient rubber gaskets. Clamping lugs, spaced 6½ in. apart around the circumference of the pipe, press the gaskets into a sealing relationship between the pipe and the coupling. These gaskets are 2½ in. by ¾ in. in section, and are partially wedge-shaped. Welded to the inside of the coupling ring (1½ in. by 2 in. section) are two bevel-faced anchor rings. The bevelled face of the rubber gasket ring is forced against the bevelled face of the anchor ring by the C-shaped cast lugs and through bolts.

The specifications required that full-sized sections of the penstock should be tested, in conjunction with the coupling, at 150 lb. per sq. in. water pressure for expansion and contraction, and articulation and offset between the pipes, in such a manner as to duplicate actual penstock conditions.

Testing the prototype coupling was, in itself, a project of sizeable proportions and required some $40,000 worth of special apparatus. The test device consisted essentially of two short lengths of 24 ft. 3 in. O.D. by 1 in. thick steel pipe representing the ends of the penstock sections being coupled, and a smaller diameter inner steel pipe or sleeve on which these were mounted. To facilitate test handling, the inner sleeve was set up with its longitudinal axis in the vertical position, and was made somewhat longer than the combined length of the 24 ft. 3 in. O.D. pipe end sections. The lower pipe end section was welded watertight to the inner sleeve, while movement of the upper pipe end section was allowed for by a rubber seal between it and the inner sleeve. This seal or gasket permitted the upper pipe end section to slide or slip in the longitudinal direction with respect to the sleeve and the fixed lower pipe.

Flexing movement was produced by 84 hydraulic cylinders attached between the inner sleeve and the upper pipe section and equally spaced around their upper periphery. In the first stage of the test, the upper

Fig. 2. One of the couplings in process of installation at Garrison Dam

pipe section was moved back and forth 1½ in. in relation to the lower pipe section while water under a pressure of 150 lb. per sq. in. was introduced into the annular space between the coupled end sections and the inner sleeve. For the second stage of the test, half of the hydraulic cylinders were compressed while the other half were extended to produce an angular misalignment of some 39 minutes between the axes of the two pipe end section under full water pressure. Repeated flexing cycles under varying pressures were recorded over a four-day test period with no indication of fracture or leakage of the coupling. This engineering feat is certainly a tribute to a company which has applied much initiative and ingenuity to improving piping techniques.

The coupling under test is shown in Fig. 1. Fig. 2 depicts a coupling being placed over one of the penstock sections at Garrison Dam. The installing contractor mounts the coupling over one of the sections of penstock pipe outside the tunnels that run through the body of the dam. Then he trucks the pipe and coupling down from the work area to the concrete-lined tunnels, and mates the coupling to a section already in place. There is barely 2 ft. between the outside diameter of the pipe and the inside diameter of the concrete-lined tunnel.

Kaplan-type Instructional Turbine

A new and interesting addition to the range of instructional water turbines for universities and technical colleges is being manufactured by Gilbert Gilkes & Gordon Limited, of Kendal, and is illustrated in Figs. 1 and 2. This is of the Kaplan type and has an 8 in. dia. adjustable-blade runner and hand-operated guide-vanes. The propeller blade angle is set when the turbine is stationary by means of a handwheel at the end of the shaft, but the guide-vane setting can be adjusted while the turbine is running. Two external guide-vanes are provided, moving with the guide-vanes themselves, thus enabling students to see the shape of the water passage and to measure the opening. A series of tests is run with one setting of the runner blades and varying guide-vane openings; the turbine is then stopped, the runner blade setting is altered, and another set of tests run.

The turbine has an output of about 4·5 h.p. when working on a net head of 10 ft., but is suitable for operating on any head from 6 to 15 ft. It can be supplied with water from an axial-flow or centrifugal pump or from an overhead tank. About 1,500 to 2,000

Fig. 1. Gilbert Gilkes & Gordon Kaplan-type instructional turbine

Fig. 2. The turbine with the Kaplan runner and guide-vane unit withdrawn

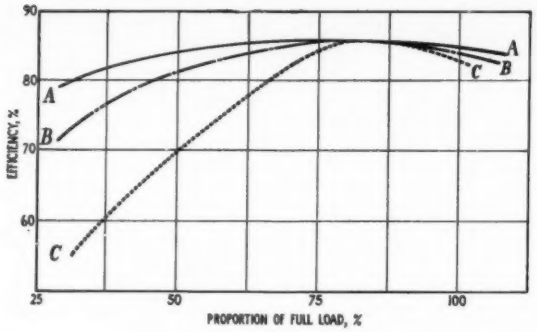


Fig. 3. Test curves obtained with the instructional turbine

gallons per min. of water is required according to the head upon which the turbine is working. The power is measured by a delicate Prony brake with a water-cooled drum, and a tachometer and pressure gauge are included as a part of the machine.

An interesting feature is the ease with which the turbine can be opened up to show the relative setting of the runner and guide-vanes. This is illustrated in Fig. 2. The makers recommend that a lifting beam and block should be permanently mounted some 7 ft. above the turbine base. It only takes about 15 mins. to take off the essential nuts, screws, etc., and the complete guide-vane and runner assembly can then be lifted clear of the casing.

Efficiencies obtained are shown in Fig. 3. Curve *A* indicates the efficiency as a full Kaplan turbine in which the runner blades and guide-vanes are adjusted together to give the maximum efficiency at any power. Curve *C* indicates the efficiency when the set is run as a fixed blade propeller turbine and curve *B* the efficiency obtained when the guide-vanes are set full open and the angle of the runner blades is adjusted according to variations in the load.

Fig. 4 is a sectional elevation of the working parts.

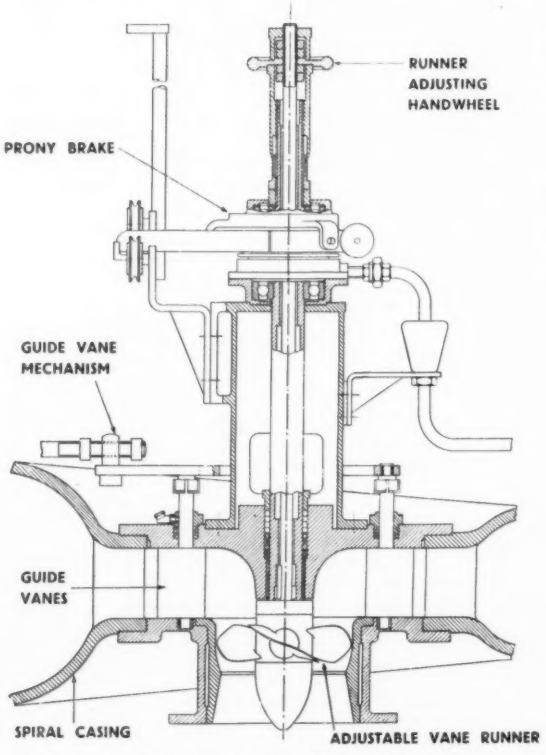


Fig. 4. Section through the G.G.G. Kaplan-type instructional turbine

Protective Equipment for Powerhouse Crane Leads

The British Thomson-Houston Co. Ltd. have designed an equipment to protect against hazards arising from the breakage of a bare overhead conductor feeding an overhead crane. Normally, in the event of such a breakage, there is a possibility that an appreciable length of live-conductor may fall down with consequent danger to life and damage to equipment at ground level. The scheme is essentially a simple one and involves a minimum of parts, and consists fundamentally of an impedance network which is connected in star to each end of the three-phase "down-shop" leads. The two star points are connected through a rectifier feeding a sensitive relay. Normally, with all three conductors intact, the two star points are at the same potential, and consequently there is no current-flow through the relay. Should one conductor, break, however, the two networks immediately become unbalanced and a voltage exists between the two star points; current flows, and energises the relay.

A normally-closed contact on the relay is connected in series with the under-voltage coil normally present on the feeder circuit-breaker; thus the relay in opening causes the circuit-breaker to trip and interrupt the supply to the crane-wires.

The essential equipment is housed in two small die-cast boxes appriximately 7 in. × 7½ in. × 5 in. deep. The Type CPS 298A unit accommodates the first network together with three fuses, the sensitive relay, rectifier unit and condenser, and the Type CPS 298B unit houses the second network, three fuses, and a test switch.

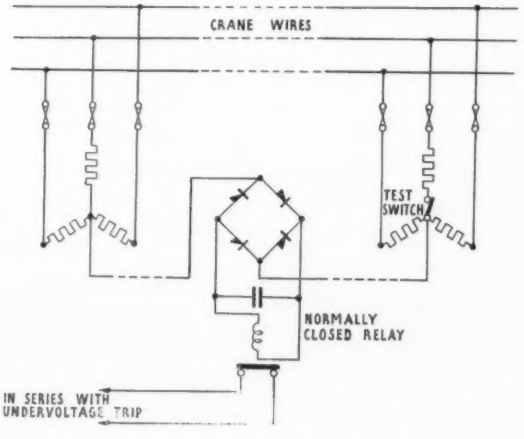


Fig. 1. Circuit diagram of B.T.H. protective equipment for crane leads

Abstracts from the World Technical Press

Underground Power Plants

The possibilities of establishing underground power stations in Germany are examined and discussed, together with the various patterns of layout, their advantages and disadvantages. Also, excavation methods are reviewed. In dealing with the constructional problems involved, the author describes briefly a project for a pumped-storage plant worked out in connection with some major development contemplated in South Germany. The appended diagram shows the general arrangement; the underground housing for the equipment is excavated in a hill standing between two storage reservoirs *A* and *B*, fed by natural flow, acting alternately as primary and secondary reservoirs in the following manner:—

1. Reservoir *A*, which has a large utilisable capacity affording perennial storage, discharging through the turbine to *B* (weekly storage), from which a possible head of 200 m. is available;

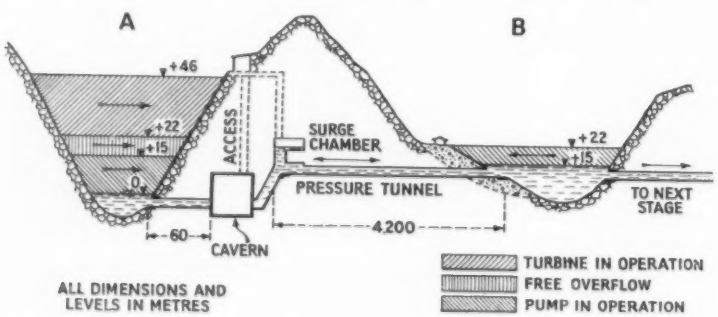
2. Free overflow from *A* to *B* of a determined volume of power water necessary to the downstream stage of the development;

3. Pumping from *A* to *B* of power water to utilise the perennial storage in *A*;

4. *B* being too small to accommodate flood discharge, pumping water back from *B* to *A* at flood periods.

From the mechanical-engineering aspect, the pumping problem can be solved either with the help of turbo-pumping sets, or by means of a turbine-pump if lower coefficients of efficiency can be tolerated during the transition stage. This is actually the case in the project instanced, as the main generating centre of the development is to be sited downstream of *B*.

As a site, the vicinity of *A* is undoubtedly the best choice, the geological conditions there (dolomitic rock) being much more favourable than near *B* (debris). In addition, the setting up of the plant close to *B* would involve a considerable amount of extra expenditure—on the one hand, the pressure tunnel, 4,200 m. in length and about 5 m. in diameter, would have to be designed for the maximum pressure, while, on the other, pumping over to *B* down to the lowest admissible level in *A* would require for the surface power station very deep foundations as well as a supply tunnel laid at a corresponding depth. These considerations led to the locating of the pumping plant close to *A*, with a short supply tunnel and a tailrace tunnel with little inside pressure, a surge tank being inserted between the latter and the pumping plant. (Dipl. Ing. Siegfried W. Markl, *Die Wasserwirtschaft*, Vol. 43, No. 4, January 1953, p. 94, 3 pp., 3 ff.)

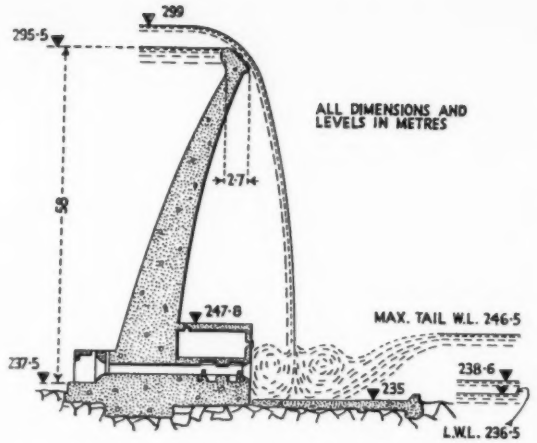


Couesque Hydro-Electric Plant

This article describes the new Couesque installation which constitutes, downstream of the Sarrans and Brommat plants, a further stage in the general development of the Truyère.

The dam (see opposite) is in a narrow and fairly symmetrical granitic gorge 6 km. upstream of the confluence of the Truyère with the Lot, and creates a reservoir of 56 million cu. m. with a utilisable capacity of 20 million cu. m. corresponding to a layer 9 m. in height. The main characteristics of this thin cantilever arch are as follows: Maximum height, 60 m.; length at crest, 175 m.; thickness at the top, 2·70 m. and at foot 11·45 m. The crest forms an arc of 114°, with a radius of 88 m., and serves as a spillway throughout its length; the arc at the level of the stilling pool has an opening of 111° and a length of 80 m. A small dam of the same type, erected on the Goul, diverts the water of this right-bank tributary of the Truyère to the Couesque reservoir by means of a free-flow tunnel 3·4 km. in length and 2·88 m. in diameter, discharging 15 cu. m. per second just above the normal storage level, about 1 km. upstream of the main dam. With a view to improving the reliability of the generating plant, twin intake works are provided on the right bank, immediately upstream of the dam, with conical connections inserted between the screens (15 × 11 m.) and the intake roller gates (3·90 × 5·0 m.). The latter are operated by oil-pressure servomotors, and can be closed either by remote control from the power house, or by hand, or automatically in cases of emergency. Behind the gates, wells 3·9 × 2 m. act as air intakes and give access to the power tunnels, which present the following features: Horseshoe cross-section 5·8 × 5·0 m., i.e., 23 4 sq. m.; length, 200 and 170 m. respectively; gradient, 12 mm. per m. The two tunnels are spaced 16·5 m. apart, and are designed for a discharge of 67 cu. m. per second each. The two sets of intake works are designed to cope each with a discharge of 67 cu. m. per sec. below the lowest utilisable level of the reservoir. The cylindrical surge tanks between the power tunnels and the penstocks, partly built above ground, are of the asymmetric-constriction type, with top expansion chambers 9·5 m. diameter by 6·4 high, and are designed to meet the emergency of a sudden total stoppage at maximum storage level. The bottom chambers, 7 m. in diameter, ensure starting from 0 to 100 per cent. of the full load with the reservoir at its minimum utilisation level.

The penstocks are laid throughout in tunnels, the 30 cm. space between a penstock and its tunnel being filled with concrete, because



of the deficient quality of the overlying rock. The steel shell is calculated to withstand the full stress of the water pressure. Each penstock is 4·2 m. in diameter, and consists of 3 m. long sections varying in thickness from 11 to 17 mm., fitted together on site by welding from inside, and reinforced by 100 mm. angle-section rings to meet the stresses which occur when empty.

The power house stands 300 m. downstream of the dam in a recess cut in the hillside so as to leave sufficient room for flood discharge; it has a ferro-concrete substructure, and a superstructure consisting of a ferro-concrete outer shell backed by a steel skeleton. The machine hall, 18 × 57 m. by 24 m. in height, is equipped with two 100-ton travelling cranes which can be coupled for handling the heaviest pieces of machinery; an erection bay and a repair shop are provided in a prolongation of the power house. There are two 32,000 kW vertical-shaft generating sets running at 214 r.p.m., as well as an auxiliary 1,000 h.p. set which ensures the return to the river of the minimum discharge prescribed, and also attends to the ancillary services. The Neyret-Beylier reaction turbines have an output of 44,000 h.p. each at a head of 58·8 m.; the Alsthom air-cooled alternators are designed for a capacity of 36,000 kVA at 15 kV. (P. C., *Le Génie Civil*, Vol. CXXXI, No. 1, 1.1.1954, p. 8, 2 pp., 3 ff.)

Taiwan (Formosa) Development

The main aim of the Chinese National Government in the hydro-electric development of Taiwan is to re-adapt to the inhabitants' daily needs power facilities from which, at the height of World War II, the Japanese had diverted a capacity of no less than 38 per cent. to aluminium production. The island's major power facilities are on the western coastal plain; there are at present in operation 24 hydro- and 8 steam-electric stations with an aggregate installed capacity of 362,145 kW, the respective share of water and steam being in the ratio of 5 to 1. With the exception of the two largest Sun-Moon Lake stations, all existing stations are run-of-river plants. All stations, except a few small ones, are connected to form an integrated system consisting of 6,400 miles of 140 kW transmission lines. No. 1 Sun-Moon Lake plant, Ta-Kuan, and No. 2, Chu-Kung, have installed capacities of 100,000 and 43,500 kW respectively. The present expansion programme contemplates the setting up of an additional aggregate capacity of 300,750 kW in 14 plants, thus almost doubling the present generating capacity of the island. The main characteristics of some of the major projects sponsored by the U.S. Foreign Operations Administration are as follow:—

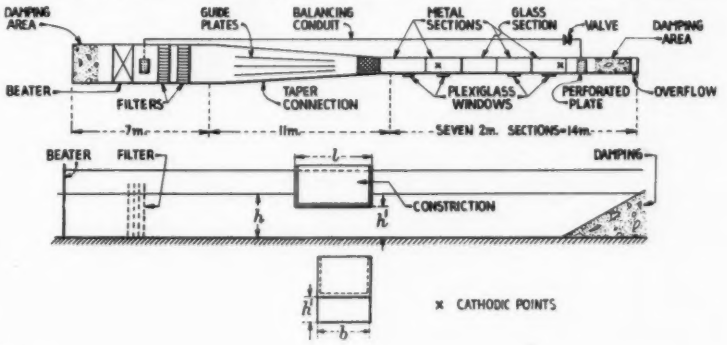
Project	Type	Units	Size	Completion Date
Li-wu Extension	Hydro	2	15,000 kW	Feb. 1954
Tien-leng	Hydro	2	26,500 kW	April 1953
Peipu Extension	Steam	2	20,000 kW	Mar. 1955
Nanpu	Steam	1	40,000 kW	May 1955
Chukeng	Hydro		100,000 kW	1957

After completion of the present projects, Taiwan will have a power generating capacity adequate to its present rate of industrialisation. Should industrial activity accelerate in the future, power expansion would not be difficult as the existing and projected plants utilise but a small slice of the available water-power resources, which are estimated at over 3·3 million kW. (T. C. Tsao, formerly Commissioner of Public Utilities for Greater Shanghai, *Electrical World*, March 29, 1954, p. 16, 3 pp., 3 ff.)

Surges in Channels

This article is a reprint of a thesis presented to the Grenoble University. In spite of the theoretical interest offered by this problem, its practical repercussions might, at first sight, appear questionable. It is emphasised, however, at the outset that permanent flow in channels is in fact an exception. Most free-flow conduits, closed or open, are subject to more or less complex and variable regimes, and their behaviour is occasionally complicated by the presence of a constriction creating a pressure section between two free-flow stretches. The practical importance of such a constriction in an otherwise open channel cannot therefore be disregarded.

A full description is given of the experimental model schematically shown in the diagram; the arrangement of the constriction in the model could hardly be simpler, consisting as it does of a box, open at the top, which can be moved vertically so as to reduce or increase the cross-section of the pressure section. We have here to deal with a very special case which, in addition, is reduced to its simplest expression, the variable flow being brought about by an induced surge or wave. The theoretical



and experimental results obtained constitute, however, valuable bases for closer and more general research in this direction. (Enzo-Oscar Macagno, *La Houille Blanche*, Vol. 9, No. 1, January-February 1954, p. 10, 27 pp., 26 ff.)

Hydraulic Jump as Energy Dissipator

Proceeding from the theory of the hydraulic jump, this paper deals with the possibility of its application to energy dissipation below overflow dams and in other hydraulic works. (C. S. Parthasarathy, B.E., *Indian Journal of Power and River Valley Development*, Vol. IV, No. 1, January 1954, p. 21, 7 pp., 7 ff.)

CLASSIFIED ADVERTISEMENTS

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